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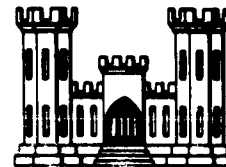
### GROUP SHELTEP INVESTIGATION

Task 8A72-04-001-31

(Office of Civil Deferse Research Project No. 1702)

29 October 1962

## U. S. Army
## Engineer Research And
## Development Laboratories
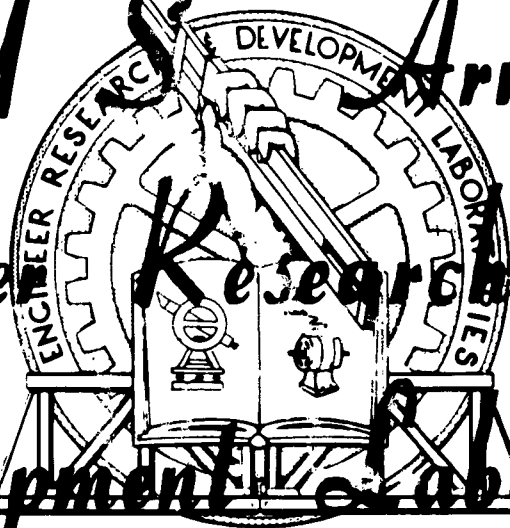
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U. S. ARMY ENGINEER RESEARCH AND DEVELOPMENT LABORATORIES
FORT BELVOIR, VIRGINIA


GROUP SHELTER INVESTIGATION


Task 8A72-04-001-31
(Office of Civil Defense Research Project No. 1702)


29 October 1962


Prepared by

Richard M. Flynn
Demolitions and Fortifications Branch
Military Department
U. S. Army Engineer Research and Development Laboratories
Fort Belvoir, Virginia

THE VIEWS CONTAINED HEREIN REPRESENT ONLY THE
VIEWS OF THE PREPARING AGENCY AND HAVE NOT
BEEN APPROVED BY THE DEPARTMENT OF THE ARMY.

PREFACE

The investigation covered by this report was conducted as a result of Office of Civil Defense Work Order No. OCD-08-62-53, Research Project No. 1702. A copy of the work order is included in Appendix A.

The following engineers were in charge of work in each of the following five divisions:

a.  Timber - Edward P. Leland.

b.  Metal - Malcolm K. Dale.

c.  Metals and Timber - John D. Morelli.

d.  Concrete - Stanley E. Woell.

e.  Plastics - John R. Fisher (DeBell and Richardson, Inc.).

Various engineers and technicians assisted. Richard M. Flynn, Chief, Fortifications Section, Demolitions and Fortifications Branch, Military Department, provided general direction and coordination of the project and prepared the final report based on reports submitted by the above personnel. These reports appear as appendices to this report.

CONTENTS

# SUMMARY

Office of Civil Defense Research Project No. 1702 requires that investigation be made into the possibilities of designing group shelter elements to enable unskilled people with only light equipment to erect low-cost group shelters. Work on project requirement was divided into four parts consonant with materials: timber, metal, concrete, and plastics. Work on timber, metal, and concrete was performed at USAERDL. Work on plastic was performed under contract with DeBell and Richardson, Inc., Hazardville, Connecticut.

The report concludes:

a.   It is possible to design and successfully develop group shelters that can be erected by unskilled groups of people.

b.   Such shelters can be erected with only hand tools and easily constructed expedient equipment such as "A" frames.

c.   Ordinary engineering materials can be used in the shelters' construction.

d.   The material cost based on a 60-man shelter will not exceed about $80 per man.

e.   Further investigation and field testing will be necessary to fully and accurately evaluate construction techniques and costs relative to a specific design.

f.   The development of plastic design 1 is desirable since it offers the potentiality of the most simple construction techniques together with costs comparable to the other engineering materials.

# GROUP SHELTER INVESTIGATION

## I.  INTRODUCTION

1.  Requirement.  On 19 February 1962, USAERDL received from the Office of Civil Defense Work Order No. OCD-08-62-53, Research Project No. 1702.  This work order required that USAERDL "explore the possibilities of designing group shelter elements so that shelters can be installed and equipped by unskilled groups of people in a manner equivalent to the current 'do-it-yourself' approach to low-cost family shelters" and further stated that "consideration shall be given to the need to avoid the use of skilled labor and also the need to avoid the use of special hard to obtain equipment such as heavy lift gear."  This requirement was, to a certain extent, modified by conference of 4 April 1962 between Marlow Stangler, Office of Civil Defense, and R. M. Flynn, USAERDL, at the former's office in the Pentagon.  According to Mr. Stangler, emphasis was to be given to erection techniques and costs.  It was also understood that this project would not be concerned with excavating work, equipment, or various appurtenances that would be installed in the shelter.  The project was to be concerned only with the bare shell of the structure and its erection techniques and cost.

2.  Approach.  There are four feasible basic engineering materials with which the shelters could be constructed:  timber, metal, concrete or masonry, and plastic.  The investigation was divided into four sections consonant with these materials.  The investigation in relation to timber, metal, and concrete materials was pursued within USAERDL, and Contract DA-44-009 Eng-4580 was let to DeBell and Richardson, Inc., Hazardville, Connecticut, for investigation of plastic material.  Work on the four approaches is reported separately in Appendices B through F.  Appendix D also presents a low-cost design using a combination of metal and timber in its construction.  Each appendix includes a reference bibliography and the specific design drawings in reduced size.  Design computations for the timber, metal, and concrete structures are presented in Appendix G.  Design calculations for plastic design 1 are included in Appendix F.

## II.  DISCUSSION

3.  Analysis of Investigation.  During the early part of the investigation, visits were made to the Waterways Experiment Station, Vicksburg, Mississippi, and to the University of Illinois, Urbana, Illinois, for consultation with Dr. Nathan M. Newmark.  In the

timber, metal, and concrete designs of this investigation, consideration was given to Dr. Newmark's work. In conformance with the verbal instructions of the Office of Civil Defense, emphasis was placed on erection techniques and costs. However, to do this, definite structural designs had to be considered and techniques and costs aligned with them. The designs were based on considerations of the most simple erection techniques with practically no equipment requirements and minimum fabrication and procurement costs.

In addition to considerations of simplified erection techniques and low procurement costs, other principal considerations in design determination were feasible protection against radiation and thermal effects and practical possibilities of construction effort. Blast resistance was considered to a limited extent only. To have blast resistance in the structural design adequate for all situations would give the structural components such great weight that heavy equipment would be necessary to move them during erection. Such equipment is prohibited by the project requirements.

The assumption was made that the nuclear effects against which the shelters would afford protection would be from megaton weapons. It seemed reasonable to assume that the enemy would almost entirely use megaton weapons against targets within the continental limits of this country. It was assumed that these weapons would in most cases be between 20 and 100 MT in size. The resulting blast pressures would not be in the dynamic range but would be to a certain extent in the static range. Under such conditions, adequate blast protection could not be included in the structural design without contravening the basic requirements of the project (apropos to unskilled personnel, minimum equipment, and low cost). Therefore, to insure that the structural components would be manhandable and thereby obviate the need for heavy equipment, the structures were designed for a maximum of 23.5-psi static pressure. A 3.5-psi value represents the static loading of 5-foot soil cover which is the greatest depth of soil cover contemplated. Therefore, the structures were designed to take a further 20-psi static load that may be imposed by blast pressure. This design loading is about the maximum possible without using heavy equipment. Table I gives data on certain of the effects of 20- and 100-MT weapons based on a 20-psi overpressure. The high intensity of thermal radiation and the comparatively negligible amount of nuclear radiation is notable. The positive blast phase is of such long duration (6 seconds) that the loading approaches a static condition; however, as shown in Fig. 1, the peak overpressure decays throughout the positive phase. As shown in Fig. 2, for a 20-MT burst, 20-psi blast pressure would be recorded at approximately 3-3/4 miles from ground zero. Also at 20-psi blast pressure, a 50-MT burst would extend to 5-1/4 miles, a

Table I.  Nuclear Effects Data

| Characteristic | 20-MT Weapon | 100-MT Weapon |
|---|---|---|
| Miles from GZ to which 20-psi overpressure extends when weapon bursts at optimum height | 4.85 | 8.3 |
| Optimum height of burst (miles)[1] | 3.2 | 5.45 |
| Slant distance to 20-psi range (miles) | 5.3 | 10.0 |
| Prompt nuclear radiation (rem)[2] | 1 | 1 |
| Thermal radiation (cal/cm$^2$)[2,3] | 350 | 500 |
| Burst wave duration, positive phase (sec) | 5.8 . | 10.0 |
| Blast wave arrival time (sec) | 15 | 25 |
| Fireball diameter (miles) | 4.6 | 8.6 |

(1)  Burst height extends 20-psi blast level to maximum distance from ground zero.

(2)  Intensity at 20-psi blast overpressure range, based on slant distance.

(3)  Visibility - 10 miles.

100-MT to 6-1/2 miles, a 10-MT to 3 miles, and a 1-MT to 1-1/4 miles. There is some question, therefore, as to the extent of protection shelters based on the designs considered in this report would offer in areas that might be prime military targets and also densely populated.  To design the shelters for a materially greater blast resistance would, however, contravene the project's basic requirements relative to inexperienced personnel and minimum of equipment.  The table and figures are based on information from Reference 1, Appendix B.

The number of occupants for a group shelter was placed at a minimum of 20 persons and a maximum of 60 persons.  It was considered that less than 20 persons would be using family-size shelters and that construction of a shelter for occupancy greater than 60 persons might be more difficult for inexperienced personnel for

several reasons (one being control of work performance). The designs were based on 10 square feet of floor space and 70 to 80 cubic feet of air volume per individual.

The shelters were generally designed with an interior center height of not more than 8 feet and not less than 6 feet with diameter or lateral dimension between 10 and 15 feet.

A soil cover depth of 1 foot minimum and 5 feet maximum with an average of 3 feet was considered to be a practical range within the project requirements. Since it is preferable that the
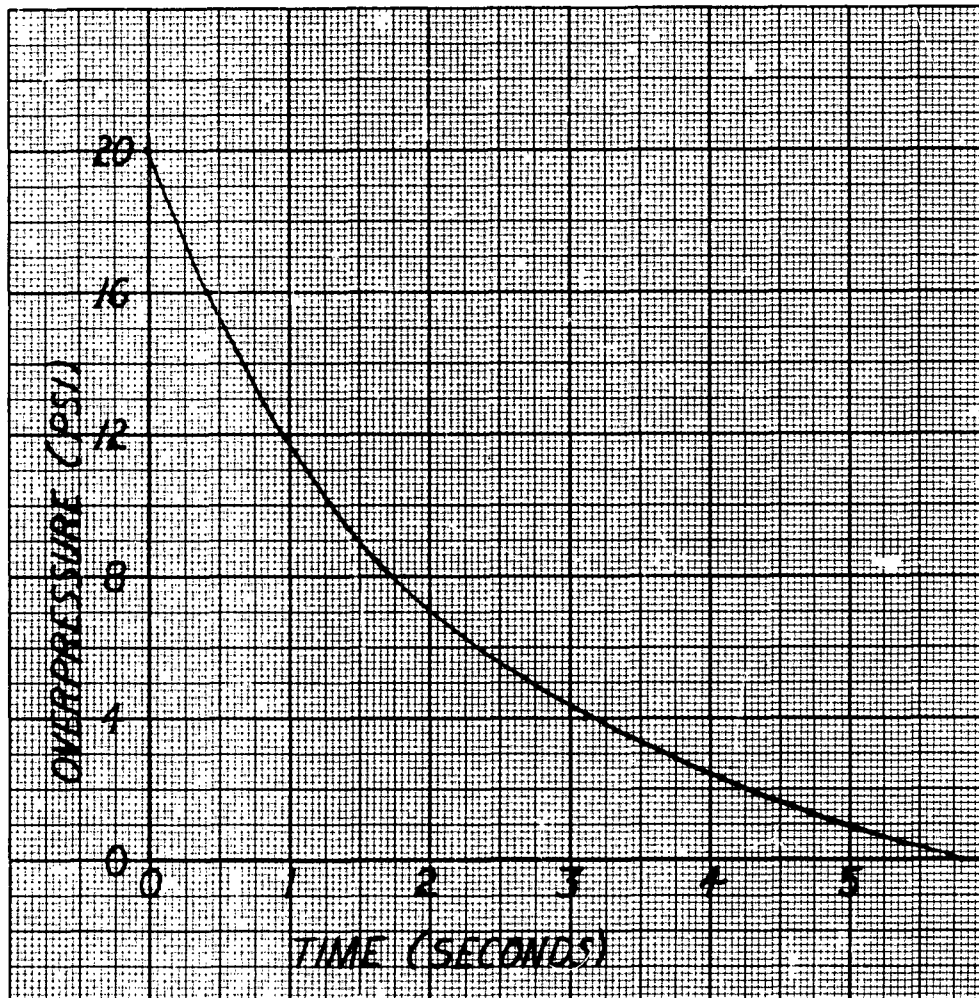


Fig. 1. Decay of 20-psi peak overpressure (20-MT optimum air burst) throughout duration of positive blast phase.
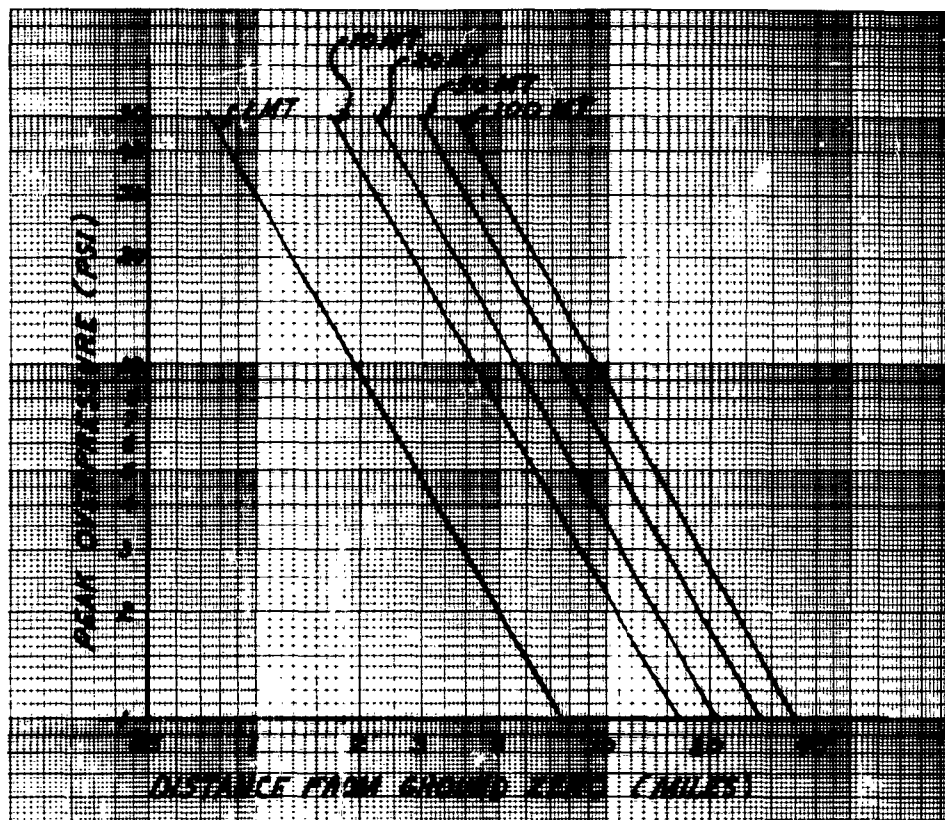
Fig. 2. Peak overpressure vs distance from ground zero for surface burst.

soil cover not extend above the ground surface, with a 5-foot maximum depth of soil cover and an 8-foot structure height, an excavation depth of about 14 feet would be required. This would be quite a deep excavation for unskilled personnel with a minimum amount of equipment. It was, therefore, assumed that this would be the maximum practical excavation depth.

The stringent limitations on structural component size and weight imposed on the designs made it possible to keep the erection techniques at maximum simplicity. Every component in all the designs is manhandable. Even the largest structural component can be carried and manipulated by a maximum of six men. Also, all components can be moved easily and placed with improvised "A" frames or slip ropes. The only equipment that would be required for the structural erection would be hand tools. No skills or extraordinary efforts will be necessary; ordinary adult intelligence and average physical strength will be sufficient for all of the tasks required.

In the backfilling of the soil, the use of a small bucket-loader would save appreciable time and effort. However, backfilling can be done manually with shovels and hand-tampers but will take a much longer time. In each of the appendices, detailed listings are given on construction procedures. None of these procedures require tools or knowledge that is not already available to the average active householder. Inexperienced groups should have no difficulty in erecting the shelters according to the plans. However, it would make understanding by inexperienced personnel easier if they were supplied manuals in which each important step in the erection operation was not only simply and explicitly explained but also graphically and pictorially represented.

In selection of the various material designs, erection simplicity and cost economy were the deciding factors. To a large extent, this circumscribed choice. For instance, it was decided not to use concrete by itself as a structural material. It seemed that concrete poured in place or the use of concrete blocks or slabs only might require more skill than could be expected to exist in an average unexperienced group. Therefore, a combination of steel structural members and concrete slabs was selected. The selection of the timber design and the metal design was not difficult. The "Post-Cap-Stringer" design in timber and a steel multiplate pipe design offered the greatest simplicity and economy in the respective materials.

In plastics, a broader research was needed because of the comparative newness of the material in group shelter design. In Appendix F, nine designs are considered. Of these nine, design 1 seems preferable. Reasons for this preference are: (a) it would require no skill to erect; (b) no equipment would be required for its erection (although its heaviest component would be quite heavy, it still could be moved and manipulated by six men); (c) erection time and requirements would be low because of the small number of components; (d) it would have very good habitability since it could be completely sealed against moisture; and (e) its cost in large production would be the lowest of the plastic designs.

On the basis of a 60-man shelter, the costs of the structural designs are:

| Timber | $80 per man |
|---|---|
| Steel | $80 per man |
| Steel (Timber) | $50 per man |
| Concrete (Steel) | $60 per man |
| Plastic | $80 per man |

These costs are approximate and cover the cost of the structural materials only. They do not cover any equipment or appurtenances such as a ventilating system or furniture. Erection labor is considered to be without charge.

The combination of steel and timber has the lowest cost. The combination of concrete and steel is about 20 percent higher. Timber, steel, and plastic structures cost approximately the same. The comparatively low cost of the plastic structure was unexpected. With the plastic structure, however, a period of development would have to ensue before the structure would be realizable.

The plastic structure, design 1, would undoubtedly be the easiest to erect. There would only be 17 component parts to the design, 12 of which would be interchangeable. All of these parts would have such distinct individual characteristics that it would instantly be evident as to how they should be erected together. Almost the whole erection procedure would consist of the bolting of these few parts together. The estimated number of man-hours required to erect this plastic structure is the lowest in comparison with the timber, metal, and concrete structures. Estimated man-hours for the erection of these structures is as follows:

| Timber | 135 |
| Steel | 500 |
| Steel - Timber | 156 |
| Concrete - Steel | 330 |
| Plastic | 120 |

These figures represent the erection of the shelter structure only and do not include excavation or backfill. Backfilling computations in Appendices B through E are based on the worst possible soil conditions where the excavation is made with a one-to-one slope. Under the best soil conditions where a vertical slope would be possible, the backfill computations could be one-quarter of those shown. However, since the personnel erecting the shelter may lack safety knowledge and training, it would be advisable to make the excavations with a one-to-one slope. The man-hours required for the backfill can vary depending on a number of factors: soil, weather, availability of a bucket-loader, and physical condition of the group members. The shoveling and compacting of the soil for the backfill is much more arduous work than that involved in the actual erection of the structure. For inexperienced personnel, the soil movement will undoubtedly be more difficult physically than will the shelter erection. The amount of soil that an average inexperienced man could move in an hour would probably vary from 1/4 cubic yard compacted and 1/2 cubic yard uncompacted under the most adverse conditions to

3/4 cubic yard compacted to 1-1/2 cubic yards uncompacted under the best conditions.

The requirements for compacting the soil and the extent of the compaction can vary depending on several factors. For an arched structure, compaction up to about two-thirds of the structure height would be advisable under most conditions. With rectangular structures, the need for compaction could vary. In any event, with inexperienced ill-equipped personnel, the degree and evenness of compacted backfill would undoubtedly be variable.

In all the designs presented in this report, the backfill amount would be approximately the same except for the concrete-steel which would require a somewhat larger excavation to afford sufficient room for the movement of the concrete slabs for the side walls. However, the erection time required for the metal structure is considerably greater than for the other materials. This is chiefly because of the large number of bolts to be placed and tightened manually in the multiplate design. If pneumatic tools were available, this time requirement could be reduced appreciably. Pneumatic tools and compressors would undoubtedly be unavailable. There is a possibility that aluminum may be available in the near future as a substitute for steel in the multiplate construction. This would enable using larger size multiplate units and would thereby reduce the bolting requirement.

4. Suggestions for Future Work. In the appendices to this report, there are presented one group shelter design for timber, metal, and concrete (steel) and several designs for plastic. All of the materials, except the plastic, are standard materials that can be purchased from suppliers' stocks. In the plastic design, a period of development will be necessary before component characteristics and fabrication procedures can be established. The other designs were based on theoretical computations and a certain amount of empirical knowledge. However, to obtain detailed and exact information on applicable construction techniques of optimum simplicity and to develop minimum cost procedures and data, it would be desirable to procure, erect, and test representative structures.

It is estimated that such further work for the development of information would cost about the same for timber, metal, and concrete-steel. The cost of the concrete-steel combination materials would be about 20 percent and the steel-timber about 35 percent less than the higher cost materials; this would not, however, materially affect the total cost. The estimated cost for further development work in the three materials would be approximately $51,250 each. A breakdown of these costs is as follows for each material:

| | |
|---|---|
| Materials | $ 5,000 |
| Labor | 16,000 |
| Engineering | 15,000 |
| General Test Expenses | 5,000 |
| Overhead | 10,250 |
| | $51,250 |

It is estimated that work on any one of the three tasks would take about 6 to 8 months.

Optimum design information would be more difficult to develop for plastic material than for the other more familiar engineering materials. In Appendix F (covering plastic structures), the selection of reinforced plastic structural materials for shelter design is discussed and manufacturing processes for producing a shelter from these materials are suggested together with an outline of the advantages and problems to be expected from designs made using the materials and processes suggested. The necessity of thoroughly investigating the problem areas involved is emphasized.

Further work should be performed to fully develop a design or designs based on the suggested designs using plastic materials. This would involve the determination of all design parameters and details implemented by complete specifications and drawings and the subsequent fabrication, erection, and testing of several structures. From this further work of development, the following necessary information would be obtained:

a.  Detailed specifications and drawings.

b.  Precise information on the best and most economical manufacturing process specifically involved.

c.  Definitive empirical information on the erection work involved and the indicated optimum erection techniques.

d.  Modifications necessary to insure the requisite stability and habitability of the structure. (This testing would comprise such features as the determination of the impregnability of the structure to wetness and contamination, the temperature and ambient air condition inside the shelter, and the effects of static and dynamic loading.)

Table II gives a breakdown of the estimated cost of each of the principal steps required in designing, preparing prototypes, and testing of two 20-man shelters of each of the eight designs proposed in this report. The first item, "Structural Design," covers

Table II.  Proposed Plastic Group Shelter Development Costs

| Principal Step | Design Number* | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
| Structural Design | $ 4,500 | $ 4,500 | $ 4,500 | $ 4,500 | $ 7,000 | $ 4,500 | $ 4,500 | $ 1,500 |
| Equipment Design | 3,000 | 3,000 | 3,000 | 3,000 | 4,000 | 3,000 | 3,000 | 500 |
| Equipment Cost and Installation | 3,500 | 3,500 | 3,500 | 3,500 | 3,500 | 13,000 | 12,000 | 1,000 |
| Mold Cost | 16,600 | 16,600 | 8,800 | 7,200 | 24,000 | 10,500 | 10,500 | 4,000 |
| Fabrication of Two 20 Person Shelters | 12,000 | 12,000 | 15,000 | 13,000 | 18,000 | 12,000 | 11,000 | 6,000 |
| Installation of Two Shelters Underground | 1,500 | 1,500 | 1,500 | 1,500 | 3,000 | 2,500 | 2,500 | 3,000 |
| Shelter Gasket Problems | 3,000 | 3,000 | 3,000 | 3,000 | 1,000 | 3,000 | 3,000 | 1,000 |
| Shelter Testing | 5,200 | 5,200 | 5,200 | 5,200 | 4,000 | 5,000 | 5,000 | 4,500 |
| Supervision and General Engineering | 23,000 | 23,000 | 23,000 | 23,000 | 27,000 | 23,000 | 23,000 | 14,000 |
| Overhead | 18,075 | 18,075 | 16,875 | 15,975 | 22,855 | 19,125 | 18,625 | 8,875 |
| Total | 90,375 | 90,375 | 84,375 | 79,875 | 114,355 | 95,625 | 93,125 | 44,375 |

| *Design | Figure |
|---|---|
| 1 | 3-6 |
| 2 | 7 |
| 3 | 11 |
| 4 | 12-14 |
| 5 | 15-17 |

consultation, design calculations, specifications, drawings, and assembly and erection details. The item "Equipment Cost" covers the specific engineering and planning required for the selection, arrangement, and construction of special manufacturing equipment required plus the purchase of standard equipment that is applicable and the installation of both. "Mold Cost" is self-explanatory. "Fabrication of Two 20-Man Shelters" includes the preparation of prototype shelters from reinforced plastic components to include an entryway and a ventilating system. "Installation Cost" covers excavation, shelter erection, and backfilling. "Testing" would consist of determining water leakage, interior air volume and temperature, and static dynamic loading. "Shelter Gasket Problems" covers the work that may arise in making all points impervious.

The estimate on design 8 includes work only on the plastic elements of the structure. Approximately $10,000 for necessary concrete work should be added to this.

Design 9 is not included in the tabulation since this design would require a certain amount of basic development to ascertain its feasibility. This would involve the development of a low-cost lightweight casting compound of plastic materials which might be employed by inexperienced personnel at the shelter installation site for the casting of sectional shelter components. The development project involved would be in two phases. The first phase would consist in the formulation and testing of mixtures to determine the feasibility of the concept. This would cost approximately $20,000. The second phase would be undertaken only after the success of the first had been assured. It would involve the development of the casting process, the designing of modular structural units, the preparation of molds, and the fabrication of a sufficient number of units for testing. This phase would cost approximately $30,000. If this development proved successful, it is anticipated that it would be followed by a project similar in objectives and scope as those estimated in the designs tabulated in Table II. The purpose of this further project would be to design a shelter specifically for the lightweight, low-cost casting material and to prepare shelters and fully evaluate them and the the techniques involved.

Development of design 1, the most preferable of the nine plastic designs, would cost approximately $90,000.

Serious consideration should be given to the use of reinforced plastic in shelter construction. Its use has a number of favorable potentialities. A prefabricated reinforced plastic structure such as design 1 could undoubtedly be erected much faster by unskilled and unequipped people than would be possible with any

other structural material. Such a structure could probably be made more habitable for a longer period of time than might be possible with other construction material. Structural strength of reinforced plastic can be comparable to that of other engineering materials. In large production, the cost of a comparable plastic design should not exceed those of most other structural material designs. In the fabrication of reinforced plastic components, ingredients such as fiberglass and polyester resin would be available in large supply sufficient for all requirements; it is anticipated that this would even be true under wartime conditions. There are also now available a large number of fabricating facilities throughout the country. A list of these facilities is given in Appendix H. These numerous facilities located throughout the country would facilitate the expediting of an urgent program and simplify the problem of distribution.

## III. CONCLUSIONS

5. <u>Conclusions</u>. It is concluded:

    a.    It is possible to design and successfully develop group shelters that can be erected by unskilled groups of people.

    b.    Such shelters can be erected with only hand tools and easily constructed expedient equipment such as "A" frames.

    c.    Ordinary engineering materials can be used in the shelters' construction.

    d.    The material cost based on a 60-man shelter will not exceed about $80 per man.

    e.    Further investigation and field testing will be necessary to fully and accurately evaluate construction techniques and costs relative to a specific design.

    f.    The development of plastic design 1 is desirable since it offers the potentiality of the most simple construction techniques together with costs comparable to the other engineering materials.

APPENDICES

APPENDIX A


AUTHORITY


C O P Y

Work Order No. OCD-OS-62-53
Research Project No. 1702

WORK ORDER
Between
DEPARTMENT OF DEFENSE, OFFICE OF CIVIL DEFENSE
and
DEPARTMENT OF THE ARMY, OFFICE OF THE CHIEF OF ENGINEERS


Department of the Army
Office of the Chief of Engineers
Washington 25, D. C.

Attention: Director of Research and Development

By virtue of Executive Order 10952 dated July 20, 1961, an order is
hereby placed with your office for furnishing the following services
to the Office of Civil Defense:

In consultation and cooperation with the Department of Defense,
Office of Civil Defense, the Department of the Army Office of the
Chief of Engineers, shall, in accordance with DA, OCE Technical
Service proposal of 22 November 1961, explore the possibilities of
designing group shelter elements so that shelters can be installed
and equipped by unskilled groups of people in a manner equivalent
to the current "do-it-yourself" approach to low-cost family shelters.
Consideration shall be given to previous work for the Department of
the Army by Dr. Nathan M. Newmark, University of Illinois, and of
possible modification of the Armco Multiplate Steel-Arch technique.
Consideration shall be given to the need to avoid the use of skilled
labor and also the need to avoid the use of special hard to obtain
equipment such as heavy lift gear. Five copies of interim reports
on significant findings and 500 copies of the final report shall be
furnished to OCD. The services for which funds are made available
under this Work Order are to be completed on or before 30 June 1962.

16

Funds in the amount of $50,000 will be reserved on our records on a reimbursable basis to cover the cost of work performed. Reimbursable billings shall be forwarded to the Comptroller, DOD, OCD, Battle Creek, Michigan, siting Appropriation 4320100 and Account 02/52/36030/9/72033.

If this order is acceptable, please sign and return three copies to the Contract Division, DOD, OCD, Battle Creek, Michigan. The original is for your retention.

DEPARTMENT OF DEFENSE                    DEPARTMENT OF THE ARMY
OFFICE OF CIVIL DEFENSE                   OFFICE OF THE CHIEF OF ENGINEERS

BY /s/ Charles T. Westcott          BY /s/ Walter H. Spinks
      CHARLES T. WESTCOTT              WALTER H. SPINKS

Title  Contracting Officer          Title  Comptroller

APPENDIX B                                    Exhibit 1

TIMBER STRUCTURES

by

Edward P. Leland

1.  General.  The timber structure considered in this section
is based on a span of 10 feet and a floor area of 600 square feet.
The structure is designed to support a maximum 5-foot soil cover.
This thickness of soil cover represents a static loading of 3.5 psi.
The design also includes the capability of enduring a 20-psi peak
blast overpressure with a long-duration positive phase.  The struc-
ture is designed to shelter 60 persons.

2.  Design Stresses.  Since the sole purpose of the timber is
to support the load acting on the structure, Stress-Grade lumber has
been specified.  In addition, the large quantities of board feet re-
quired for a national shelter system necessitate that Southern Yel-
low Pine and Douglas-Fir be employed because these two woods are the
most plentiful.

In Reference 2, page IV, Douglas-Fir is shown in two types
although allowable stresses are the same.  Therefore, it is not
necessary to specify coast type Douglas-Fir.  Of the several grades
of stress-grade Douglas-Fir available, varying from commercial deck-
ing or 1200f industrial to dense select structural, it is necessary
that the dense construction grade be the minimum requirement.

Southern Yellow Pine is a better wood overall than Douglas-
Fir since its best grades are superior to the best grades of Douglas-
Fir; however, the poorest and intermediate grades of both are about
the same.  The available grades of Southern Pine (Reference 2, page
VI) for 5-inch-thick and above material vary from No. 2 stress rated
up to dense structural 86.  It is necessary that the minimum grade
requirement for Southern Pine be No. 1 dense stress rated.  In addi-
tion, another construction wood which would be suitable is larch
(dense construction grade or better).  The other stress-grade woods
(Reference 2) are inferior to the ones specified above.

In Reference 3, pages 58-9, it is stated that the allow-
able stresses in current timber specifications may be multiplied by
4 if the timber specified is of stress grade.  Under instantaneously
applied loads, timber develops a strength 2 or more times the usual

static strength under such loads even if the load duration is 1 or 2 seconds. A comparison of test data and the allowable working stresses for Southern Pine and Douglas-Fir are shown in Table III. Also included are the design stresses employed for this dynamically loaded-type structure. In some instances, the power of 4 factor was reduced because of the test data (compression parallel to grain) and (compression perpendicular to grain). The design stress for horizontal shear was increased by a factor greater than 5 since the allowable stress is such a small part of the test strengths. The strength of timber is materially affected by moisture content. Therefore, it is necessary that the lumber be dried before it is placed in the structure.

3. <u>Design Type</u>. The type of design used is the Post-Cap-Stringer construction utilized in Reference 4. This design is economical in material and can be constructed by inexperienced personnel. The roof stringers support the soil cover. The stringers in turn are supported by longitudinal caps which are supported at their ends by posts. Sills, same size as caps, serve as footings for the posts. These sills will obviate marked differential settlement between adjacent posts; such settlement might otherwise occur if footings were used. Side sheathing is employed to keep soil walls stable. Spreaders are used to keep the walls apart. The soil side loading is primarily a bending load on the post with some loading on the caps and sills. Scabs are used to combine the side loading on the caps and posts and transfer side loading to the spreaders. Diagonal stiffeners are used between the posts to provide stability during erection.

The horizontal portion of the entranceway employs the same design, Post-Cap-Stringer. The only variation from this design in the horizontal portion is the inclusion of longitudinal braces to transfer the horizontal soil loading from the end posts to the vertical shaft and vice versa. A variation of this design is used in the vertical portion of the entranceway. Side sheathing maintains soil stability. This sheathing is supported by vertical beams, one in each corner of the shaft. The vertical beams are continuous over two spans. Bracing at the ends and the midpoint supports the beams. The vertical beams are connected to the end posts of the horizontal portion.

The cover over the vertical shaft or hatch was designed to be separate from the hatch so as not to transfer loads to the hatch. The cover is supported by sills. Rebound of the cover during the negative blast phase is obviated by deadmen attached to the sills. The cover is hinged to one sill and is capable of being fastened to the other. The cover, with its 8 stringers, 2 beams, and 2 sidings,

Table III.  Strength Properties of Woods

| | Moisture Content[1] (%) | Specific Gravity[2] | Fiber Stress at Proportional Limit (psi) | | | Modulus of Elasticity (1000 psi) | Max. Shearing Strength Parallel to Grain (psi) |
|---|---|---|---|---|---|---|---|
| | | | Bending | Compression Parallel to Grain | Compression Perpendicular to Grain | | |
| Douglas-Fir[3] Coast-type | 38 / 12 | 0.45 / 0.48 | 4,500 / 7,800 | 3,130 / 5,850 | 440 / 870 | 1,570 / 1,950 | 930 / 1,160 |
| Southern-Yellow[3] Pine, Longleaf Type | 63 / 12 | 0.54 / 0.58 | 5,200 / 9,300 | 3,430 / 6,150 | 590 / 1,190 | 1,600 / 1,990 | 1,040 / 1,500 |
| Southern-Yellow[3] Pine, Shortleaf Type | 81 / 12 | 0.46 / 0.51 | 3,900 / 7,700 | 2,500 / 5,090 | 440 / 1,000 | 1,390 / 1,760 | 850 / 1,310 |
| Southern Pine or Douglas-Fir,[4] design stresses | 12 specified | 0.56 | 6,000 | 4,800 | 1,100 | 1,760 | 640 |
| Allowable Stresses, Douglas-Fir[5] Dense Construction Grade, P&T | -- | -- | 1,500 | 1,400 | 455 | 1,760 | 120 |
| Allowable Stresses, Southern Pine[5] No. 1 Dense SR, 5" thick and up | -- | -- | 1,600 | 1,500 | 455 | 1,760 | 120 |

1. The results in the first line for each species are from tests of green material; those in the second line are from tests of seasoned material adjusted to an average air-dry condition of 12% moisture content.
2. Based on oven-dry weight and volume when green or at 12% moisture content.
3. Results of tests on small, clear specimens in the green and air-dry condition (Reference 2, pp. 75-76)
4. Design stresses based on above test results and factor of four specified in Reference 3.
5. Reference 1.

weighs 220 pounds. Unless a counter-balance is provided, it would
be necessary to make this cover in at least two portions so as to
enable it to be opened and closed fairly easily.

The entrance design involves two basic assumptions: That
two 90-degree bends are desirable for radiation attenuation; and
that speed of entry is not essential thereby permitting use of a
vertical section with ladder. The usual fallout shelter design
provides only one 90-degree turn in the entranceway; however, it
is believed that the extra turn will reduce the radiation entering
through the entranceway to a very small amount. Cost estimates are
given for both a two-turn and a one-turn entranceway. A vertical-
shaft entrance design requires less construction effort than a ramp
type, and it also gives greater radiation protection. The vertical
shaft has been sized at 3 feet square. The horizontal portion has
been sized at 3 feet wide and 5 feet high. The height figure of 5
feet will necessitate that a person bend over when walking through
the entranceway but will not make it necessary to crawl.

A 200-pound beam is the maximum weight beam that can be
manhandled into position during erection. Therefore, beam sizes
were kept below 200 pounds, with the exception of the entrance bulk-
head where a major fabrication problem occurs unless horizontal end
beams are used. Because of its length, this beam weighs 250 pounds.
There are only two of these beams, however, and only one has to be
lifted into position.

The end bulkheads of the main shelter offer a major design
problem. One solution is to transfer the loading on the ends to-an
axial load on the caps and sills. By this method, end posts and end
beams are used to transfer soil loading from end sheathing. It is
believed that this method is safe considering the soil loading re-
sisting the tendency for the caps to slip out of position. Another
solution is to make a natural soil bulkhead (Reference 5, page 32).
In this instance, soil is sloped at one to one with expected damage
to consist only of minor sloughing of the slope. In Reference 6,
page 102, it is pointed out that this endwall design has not been
tested against nuclear effects. Reference 6 also notes that the top
of the earth fill forming the bulkhead must be at least 2 feet with-
in the structure. The length of the basic structure must be extend-
ed so that the structure will cover the soil bulkhead. A third
method, not utilized, is to employ deadmen with bulkheads separate
from the shelter (i. e., bulkhead loading would be resisted by the
deadmen and would not be imposed upon the remaining shelter structure)

The entranceway fits into an opening in the entrance bulk-
head of the main shelter, but in order to prevent one structure from

distorting the other under load, no connections are provided between the two.

4.  Timber Treatment. The expected life of this type of shelter would be at least several years. Since the timber members will be in direct contact with soil, it is obvious that deterioration of the timber will occur unless it is treated. Various chemical treatments will enhance the life of timber. Creosote and petroleum oils fortified with chlorinated phenols, principally pentachlorophenol, or with copper naphthenate are one type of wood preservative (Reference 7, pages 399 and 404). Although very effective against decay fungi and harmful insects, these preservatives are skin irritants, have objectionable odors, and leave undesirable finishes on the timber. Another type of treatment is the use of waterborne salts that are applied as water solutions.

Standard wood preservatives used in water solution include zinc chloride, chromated zinc chloride, copperized chromated zinc chloride, Tanalith (Wolman Salts), acid copper chromate (Celcure), zinc meta arsenite, ammoniacal copper arsenite (Chemonite), chromated zinc arsenate (Boliden salt), and chromated copper arsenate (Greensalt or Erdalith). These preservatives are employed principally in the treatment of wood for uses where it will not be in contact with the ground or water and where the treated wood requires painting. As a general rule, they are less resistant to leaching and do not perform so satisfactorily as the preservative oils under conditions favorable to leaching. The leaching resistance of some of these preservatives has been developed to the extent that good performance can be expected in ground contact or in other wet installations, but they are still not considered equal in effectiveness to creosote when used under such conditions. On the other hand, waterborne preservatives are generally preferable to creosote for indoor use and can give indefinitely long life where not subject to leaching (Reference 7).

Waterborne preservatives leave the wood surface comparatively clean, paintable, and free from objectionable odor. With several exceptions, they must be used at low treating temperatures (100 to 160° F) because of their instability at the higher temperatures common with preservative oils. This may involve some difficulty when higher temperatures are needed to obtain good treating results in such woods as Douglas-fir. Since water is added during treatment, the wood must be dried after treatment to the moisture content required for use (Reference 7).

Zinc chloride and chromated zinc chloride are frequently used as fire retardants for wood but at retentions higher than those used only for wood-preserving purposes (Reference 7).

All cutting, framing, and boring of holes should be done before treatment. Cutting into the wood in any way after treatment will frequently expose the untreated interior of the timber and permit ready access to decay fungi or insects. It is much more practical than is commonly supposed to design wood structures so that all cutting and framing may be done before treatment (Reference 7).

Table IV gives a comparison of the effectiveness of various treatments as compared with untreated timber fence posts. Even though inferior to the oils and oil solutions, the salts do show definite effectiveness (the poorer ones show an average life of 20 years or better) and, in some instances (Celcure, zinc meta arsenite, and copper sulfate-sodium arsenate), outstanding effectiveness. The untreated posts had an average life of 3.3 years as compared with a minimum life of 16 years for all the posts treated with zinc meta arsenite (ZMA) (Reference 7).

Generally speaking, the less moisture in wood the greater the strength. Therefore, the timber employed in this shelter should be dried thoroughly before it is installed in the shelter. If the timber is treated with preservative, it also should be dried after the treatment. Douglas-Fir requires a higher treatment temperature for adequate penetration than does Southern Pine. Although the addition of preservatives is not regarded as being deleterious to the strength of timber, it is known that strength is reduced when high treatment temperatures are necessary. Therefore, the selection of a salt preservative to be added to Douglas-Fir should be carefully made so as to make sure that very high treatment temperatures are not necessary.

5. Design Data. The design computations in Appendix G have been performed using the stresses listed in Table III. Equations employed came generally from Reference 2. Tabular data in Reference 8 simplified the mathematics. Considering the size of the dynamic loading, it was deemed unnecessary to consider the weight of the timber members themselves $(35 \text{ lb/ft}^3)$. Therefore, no allowance for this small additional load was made.

It should be noted that the design has not been entirely carried out. Minor details such as type of fastenings and hardware have not been cited. Neither has flooring been provided. The design of flooring (if desired) presents no problem. The flooring would be separate from the rest of the structure so as to obviate movement from blast (Reference 9, page 16) and would be soil supported. It is believed that the design has been carried far enough for the specified purpose. All timber is S4S. Design is based upon actual dimensions. Nominal sizes are used only to compute board feet.

Table IV. Condition of Round Southern Yellow Pine Experimental Fence Posts at Harrison Experimental Forest, Saucier, Miss., After 11½ to 16 Years of Service

| Preservative(1) | Posts in Test(2) | Preservative Form | Retention of Preservative (lb/ft³)(3) | | | | Method of Treatment | Post Conditions December 1952 | | | | | | Average Life (yr)(4) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Minimum | Maximum | Average | Standard Deviation | | Serviceable (%) | % Removed | | | Total Removed | | |
| | | | | | | | | | Decay | Decay Termites | Termites | No. | % | |
| **Posts set late in 1936 to Feb 1937** | | | | | | | | | | | | | | |
| Celcure (acid cupric chromate) | 94 | Salt | 0.75 | 1.05 | 0.92 | 0.08 | Pressure | 90.4 | 7.5 | 2.1 | --- | 9 | 9.6 | --- |
| Chromated zinc chloride | 97 | " | 0.37 | 1.33 | .87 | .19 | " | 74.2 | 14.4 | 11.4 | --- | 25 | 25.8 | 20 |
| Coal-tar creosote, grade 1 | 98 | Oil | 1.90 | 8.60 | 6.00 | 1.50 | " | 98.0 | 2.0 | --- | --- | 2 | 2.0 | --- |
| Coal-tar creosote, 50 percent; used crankcase oil, 50 percent (by volume) | 98 | Solution(5) | 1.60 | 14.80 | 5.40 | 2.20 | " | 99.0 | 1.0 | --- | --- | 1 | 1.0 | --- |
| Pentachlorophenol, 4.82 percent (by weight) in used crankcase oil | 98 | Solution | 2.90 | 9.50 | 6.70 | 1.60 | - | 100.0 | --- | --- | --- | -- | --- | --- |
| Tanalith (Wolman Salts) | 97 | Salt | .20 | .47 | .35 | .07 | " | 80.4 | 12.4 | 3.1 | 4.1 | 19 | 16.6 | 22 |
| Zinc chloride | 98 | " | .67 | 1.11 | .94 | .10 | " | 84.7 | 4.1 | 11.2 | --- | 15 | 15.3 | 23 |
| Zinc meta arsenite | 96 | " | .25 | .54 | .42 | .06 | " | 100.0 | --- | --- | --- | -- | --- | --- |
| Untreated posts (set Feb 1937) | 65 | --- | --- | --- | --- | --- | None | ---- | 3.1 | 93.8 | 3.1 | 65 | 100.0 | 3.1 |
| Untreated posts (set Nov, Dec 1938) | 33 | --- | --- | --- | --- | --- | None | ---- | 33.3 | 63.6 | 3.1 | 33 | 100.0 | 3.7 |
| Copper sulfate and sodium arsenate (set May 1941) | 99 | Salt | ---- | ---- | .35 .16 | ---- | Double Diffusion | 99.0 | 1.0 | --- | --- | 1 | 1.0 | --- |

1. Data extracted from Reference 7, Table 45.
2. Installation included 100 posts for each treatment. This number has since been reduced in some cases by fire and pilferage.
3. Based on the 100 posts treated in each group, unless otherwise indicated.
4. Average life of all untreated posts is 3.3 years; other values are estimates taken from a mortality curve. Where percentage of posts removed is 10 percent or less, no estimate on average life is given.
5. Retention values based on 97 posts.

6. Estimates. The three estimates discussed below cover timber cost, construction effort of timber portion of shelter, and placement of compacted and uncompacted backfill. In the estimates, no costs have been included for purchase of site or for excavation. No costs were included for miscellaneous hardware, nails, spikes, bolts, drift pins, etc., as these items would only effect a minor increase in the overall cost.

      a. Timber Cost. Estimates obtained for the cost of timber were as follows:

| Size (in.) | $/thousand board measure (mbm) |
|---|---|
| 2 x 4 | $135 |
| 4 x 10 | 235 |
| 5 x 5 | 307 |
| 6 x 6 | 252 |
| 8 x 8 | 288 |
| 10 x 10 | 298 |

The prices were quoted by a retailer and applied to stress-grade for Southern Pine or Douglas-Fir. The out-of-line cost on 5- by 5-inch timber was due to necessity of trimming 6- by 6-inch material. It would appear that, when the timber is obtained from a wholesale source, the average price would not exceed $250/mbm. In very large quantity procurement, the price might be lowered to $200/mbm. For the purpose of this investigation, it is assumed that the average cost of timber is $225/mbm. Bills of material for the main shelter and the entranceway computed both with timber bulkheads and with soil bulkheads are presented in Table V.

Table V. Bill of Materials (Lumber)

| Member Type | Size (in. x in. x ft-in.) | Quantity | Wt Each (lb) | Wt Total (lb) | fbm Each | fbm Total |
|---|---|---|---|---|---|---|
| (a) Main Shelter (Timber Bulkhead) | | | | | | |
| Roof Stringers | 8 x 8 x 11 - 0 | 100 | 151 | 15,100 | 58.6 | 5,860 |
| Cap | 10 x 10 x 5 - 6-3/4 | 2 | 122 | 244 | 46.3 | 93 |
| Cap | 10 x 10 x 5 - 0 | 20 | 110 | 2,200 | 41.7 | 834 |
| Cap | 10 x 10 x 5 - 5-3/4 | 2 | 120 | 240 | 45.6 | 91 |
| Sill | 10 x 10 x 5 - 6-3/4 | 2 | 122 | 244 | 46.3 | 93 |
| Sill | 10 x 10 x 5 - 0 | 20 | 110 | 2,200 | 41.7 | 834 |

Table V (cont'd)

| Member Type | Size (in. x in. x ft-in.) | Quantity | Wt Each (lb) | Wt Total (lb) | fbm Each | fbm Total |
|---|---|---|---|---|---|---|
| Sill | 10 x 10 x 5 - 5-3/4 | 2 | 120 | 240 | 45.6 | 91 |
| Post | 12 x 10 x 7 - 0 | 26 | 186 | 4,830 | 70.0 | 1,820 |
| Scab | 12 x 4 x 1 - 8 | 52 | 17 | 883 | 6.7 | 348 |
| Spreader | 6 x 6 x 9 - 4-3/4 | 26 | 70 | 1,820 | 28.2 | 733 |
| End Post | 10 x 10 x 8 - 7 | 2 | 188 | 376 | 71.5 | 143 |
| End Beam | 10 x 10 x 11 - 7 | 2 | 253 | 506 | 96.4 | 193 |
| Sheathing | 10 x 3 x 6 - 3-1/4 | 20 | 38 | 760 | 15.7 | 314 |
| Sheathing | 10 x 6 x 11 - 0 | 11 | 140 | 1,540 | 55.0 | 605 |
| Sheathing | 8 x 5 x 8 - 6 | 12 | 70 | 840 | 28.3 | 340 |
| Sheathing | 10 x 3 x 10 - 0 | 90 | 61 | 5,490 | 25.0 | 2,250 |
| Sheathing | 10 x 3 x 11 - 3-1/4 | 20 | 69 | 1,380 | 28.2 | 564 |
| Sheathing | 16 x 5 x 5 - 8 | 1 | 96 | 96 | 38.5 | 38 |
| Stiffener | 2 x 4 x 8 - 1 | 24 | $11\frac{1}{2}$ | 276 | 5.4 | 130 |
| | | | Totals: | 39,265 | | 15,374 |

(b)  Entranceway (Timber Bulkhead)

| Member Type | Size (in. x in. x ft-in.) | Quantity | Wt Each (lb) | Wt Total (lb) | fbm Each | fbm Total |
|---|---|---|---|---|---|---|
| Roof Stringer[a] | 12 x 3 x 3 - 11 | 14 | 29 | 406 | 11.8 | 165 |
| Cover Stringer | 8 x 3 x 5 - 4-1/4 | 8 | 26 | 208 | 10.7 | 86 |
| Cap | 6 x 8 x 5 - 9-1/2 | 2 | 58 | 116 | 23.2 | 46 |
| Cap[b] | 6 x 8 x 7 - 8-1/2 | 2 | 77 | 154 | 30.8 | 62 |
| Post[c] | 10 x 6 x 5 - 0 | 6 | 64 | 384 | 25.0 | 150 |
| Post[b] | 6 x 6 x 5 - 0 | 3 | 37 | 111 | 15.0 | 45 |
| Post[b] | 12 x 6 x 5 - 0 | 1 | 77 | 77 | 30.0 | 30 |
| Sill | 6 x 8 x 5 - 9-1/2 | 2 | 58 | 116 | 23.2 | 46 |
| Sill[b] | 6 x 8 x 7 - 8-1/2 | 2 | 77 | 154 | 30.8 | 62 |
| Scab | 10 x 2-1/2 x 1 - 3 | 8 | 6 | 48 | 2.6 | 21 |
| Scab[b] | 6 x 3 x 1 - 3 | 8 | $4\frac{1}{2}$ | 36 | 1.9 | 15 |
| Scab[b] | 8 x 3 x 1 - 3 | 4 | 6 | 24 | 2.5 | 10 |
| Spreader | 4 x 4 x 2 - 7-3/4 | 4 | $8\frac{1}{2}$ | 34 | 3.5 | 14 |
| Spreader[b] | 4 x 4 x 2 - 6-3/4 | 4 | 8 | 32 | 3.4 | 14 |
| Spreader[b] | 5 x 6 x 2 - 6-3/4 | 2 | 15 | 30 | 6.4 | 13 |
| Brace[b] | 6 x 2-1/2 x 3 - 0 | 4 | 9 | 36 | 3.8 | 15 |
| Brace[b] | 6 x 2-1/2 x 2 - 11 | 2 | $8\frac{1}{2}$ | 17 | 3.7 | 7 |
| Brace[b] | 6 x 2-1/2 x 2 - 7 | 2 | $7\frac{1}{2}$ | 15 | 3.2 | 6 |
| Brace | 6 x 6 x 3 - 0 | 4 | 22 | 88 | 9 | 36 |
| Brace | 4 x 4 x 3 - 0 | 8 | 10 | 80 | 4 | 32 |
| Sheathing | 10 x 3 x 5 - 7-3/8 | 8 | 34 | 272 | 14.0 | 112 |

Table V (cont'd)

| Member Type | Size (in. x in. x ft-in.) | Quantity | Wt Each (lb) | Wt Total (lb) | fbm Each | fbm Total |
|---|---|---|---|---|---|---|
| Sheathing[b] | 10 x 2-1/2 x 3 - 10-1/4 | 8 | 19 | 152 | 8.0 | 64 |
| Sheathing[b] | 10 x 2-1/2 x 3 - 9-1/2 | 8 | 18½ | 148 | 7.9 | 63 |
| Sheathing | 10 x 3 x 9 - 8-1/2 | 8 | 58 | 464 | 24.3 | 194 |
| Sheathing[b] | 10 x 2-1/2 x 7 - 8-1/2 | 4 | 38 | 152 | 16.1 | 64 |
| Sheathing | 12 x 2-1/2 x 4 - 3 | 45 | 25 | 1,125 | 10.6 | 477 |
| Sill | 12 x 4 x 5 - 6 | 2 | 55½ | 111 | 22.0 | 44 |
| Siding | 3 x 4 x 5 - 4-1/4 | 2 | 12 | 24 | 5.4 | 11 |
| Beam | 3 x 4 x 5 - 0 | 2 | 11½ | 23 | 5.0 | 10 |
| Beam | 8 x 8 x 12 - 2-5/8 | 4 | 168 | 672 | 65.2 | 261 |
| Deadman | 6 x 4 x 5 - 6 | 2 | 27.5 | 55 | 11.0 | 22 |
| | | Totals: | | 5,364 | | 2,197[d] |

### (c)  Main Shelter (Soil Bulkhead)

| Member Type | Size (in. x in. x ft-in.) | Quantity | Wt Each (lb) | Wt Total (lb) | fbm Each | fbm Total |
|---|---|---|---|---|---|---|
| Roof Stringers | 8 x 8 x 11 - 0 | 132 | 151 | 19,930 | 58.6 | 7,740 |
| Cap | 10 x 10 x 5 - 0 | 32 | 110 | 3,520 | 41.7 | 1,335 |
| Sill | 10 x 10 x 5 - 0 | 32 | 110 | 3,520 | 41.7 | 1,335 |
| Post | 12 x 10 x 7 - 0 | 34 | 186 | 6,320 | 70.0 | 2,380 |
| Scab | 12 x 4 x 1 - 8 | 68 | 17 | 1,156 | 6.7 | 456 |
| Spreader | 6 x 6 x 9 - 4-3/4 | 34 | 70 | 2,380 | 28.2 | 958 |
| Sheathing | 10 x 3 x 5 - 0 | 20 | 31 | 620 | 12.5 | 250 |
| Sheathing | 10 x 3 x 10 - 0 | 150 | 61 | 9,150 | 25.0 | 3,750 |
| Stiffener | 2 x 4 x 8 - 1 | 32 | 11½ | 368 | 5.4 | 173 |
| | | Totals: | | 46,964 | | 18,377 |

### (d)  Entranceway (Soil Bulkhead)

| Member Type | Size (in. x in. x ft-in.) | Quantity | Wt Each (lb) | Wt Total (lb) | fbm Each | fbm Total |
|---|---|---|---|---|---|---|
| Roof Stringer[a] | 12 x 3 x 3 - 11 | 14 | 29 | 406 | 11.8 | 165 |
| Cover Stringer | 8 x 3 x 5 - 4-1/4 | 8 | 26 | 208 | 10.7 | 86 |
| Cap | 6 x 8 x 5 - 9-1/2 | 4 | 58 | 232 | 23.2 | 93 |
| Cap[b] | 6 x 8 x 7 - 8-1/2 | 2 | 77 | 154 | 30.8 | 62 |
| Sill | 6 x 8 x 5 - 9-1/2 | 4 | 58 | 232 | 23.2 | 93 |
| Sill[b] | 6 x 8 x 7 - 8-1/2 | 2 | 77 | 154 | 30.8 | 62 |
| Post[e] | 10 x 6 x 5 - 0 | 10 | 64 | 640 | 25.0 | 250 |

Table V (cont'd)

| Member Type | Size (in. x in. x ft-in.) | Quan- tity | Wt Each (lb) | Wt Total (lb) | fbm Each | fbm Total |
|---|---|---|---|---|---|---|
| Post[b] | 6 x 6 x 5 - 0 | 3 | 37 | 111 | 15.0 | 45 |
| Post[b] | 12 x 6 x 5 - 0 | 1 | 77 | 77 | 30.0 | 30 |
| Scab | 10 x 2-1/2 x 1 - 3 | 16 | 6 | 96 | 2.6 | 42 |
| Scab[b] | 6 x 3 x 1 - 3 | 8 | 4½ | 36 | 1.9 | 15 |
| Scab[b] | 8 x 3 x 1 - 3 | 4 | 6 | 24 | 2.5 | 10 |
| Spreader | 4 x 4 x 2 - 7-3/4 | 8 | 8½ | 68 | 3.5 | 28 |
| Spreader[b] | 4 x 4 x 2 - 6-3/4 | 4 | 8 | 32 | 3.4 | 14 |
| Spreader[b] | 5 x 6 x 2 - 6-3/4 | 2 | 15 | 30 | 6.4 | 13 |
| Brace[b] | 6 x 2-1/2 x 3 - 0 | 4 | 9 | 36 | 3.8 | 15 |
| Brace[b] | 6 x 2-1/2 x 2 - 11 | 2 | 8½ | 17 | 3.7 | 7 |
| Brace | 6 x 6 x 3 - 0 | 4 | 22 | 88 | 9 | 36 |
| Brace | 4 x 4 x 3 - 0 | 8 | 10 | 80 | 4 | 32 |
| Brace[b] | 6 x 2-1/2 x 2 - 7 | 2 | 7½ | 15 | 3.2 | 6 |
| Sheathing | 10 x 3 x 9 - 8-1/2 | 8 | 58 | 464 | 24.3 | 194 |
| Sheathing | 10 x 3 x 5 - 7-3/8 | 8 | 34 | 272 | 14.0 | 112 |
| Sheathing | 10 x 3 x 5 - 9-1/2 | 16 | 35 | 560 | 14.5 | 232 |
| Sheathing[b] | 10 x 2-1/2 x 3 - 10-1/4 | 8 | 19 | 152 | 8.0 | 64 |
| Sheathing[b] | 10 x 2-1/2 x 3 - 9-1/2 | 8 | 18½ | 148 | 7.9 | 63 |
| Sheathing[b] | 10 x 2-1/2 x 7 - 8-1/2 | 4 | 38 | 152 | 16.1 | 64 |
| Sheathing | 12 x 2-1/2 x 4 - 3 | 45 | 25 | 1,125 | 10.6 | 477 |
| Footing | 12 x 4 x 5 - 6 | 2 | 55½ | 111 | 22.0 | 44 |
| Siding | 3 x 4 x 5 - 4-1/4 | 2 | 12 | 24 | 5.4 | 11 |
| Beam | 3 x 4 x 5 - 0 | 2 | 11½ | 23 | 5.0 | 10 |
| Beam | 8 x 8 x 12 - 2-5/8 | 4 | 168 | 672 | 65.2 | 261 |
| Deadman | 6 x 4 x 5 - 6 | 2 | 27.5 | 55 | 11.0 | 22 |
| | | Totals: | | 6,494 | | 2,658[f] |

(a) Only 6 needed when horizontal bend is removed.
(b) Eliminated when horizontal bend is removed.
(c) Only 4 needed when horizontal bend is removed.
(d) Total board measure when horizontal bend is removed: 1583.
(e) Only 8 needed when horizontal bend is removed.
(f) Total board measure when horizontal bend is removed: 2044.

In addition to the basic cost of the timber, there is a cost involved in treatment of the timber with a waterborne salt preservative. Quotes obtained from available retail sources cite $40 to 50/mbm for salt treatments. It is assumed, that in a quantity procurement, this cost would not exceed $35/mbm, a figure which has been used in this estimate.

In the timber bills of material (Table V), an additional breakdown is provided between the entranceway having two 90-degree turns and one having only one 90-degree turn. Costs of the various designs are recorded in Table VI.

Table VI. Cost Analysis of Shelter

| Bulk-head Type | No. of Turns in Entrance-way | Timber Purchase | | | | | |
|---|---|---|---|---|---|---|---|
| | | Quantity (mbm) | Salt Treatment ($/mbm) | Total Treatment Cost ($) | Untreated Timber ($/mbm) | Untreated Timber Total Cost ($) | Grand Total Cost ($) |
| Timber | 2 | 17.571 | 35 | 615 | 225 | 3950 | 4565 |
| Timber | 1 | 16.957 | 35 | 593 | 225 | 3813 | 4406 |
| Soil | 2 | 21.035 | 35 | 736 | 225 | 4740 | 5476 |
| Soil | 1 | 20.421 | 35 | 715 | 225 | 4600 | 5315 |

b.  Erection Effort. It is assumed that virtually all drift pin and bolt holes needed would be drilled into the timber prior to the salt preservative treatment. This assumption reduces the time required for erection at the site and insures that the preserving treatment would be more effective.

It is also assumed that picking up timber and carrying by hand into the excavation is part of the man-hours required to place each piece of timber. Since most of the members either act as simple beams or as columns, it is assumed that driving single nails at each point of fastening is sufficient except where toenailing is required. Multiple toenailing of columns is required.

Table VII is an estimate of erection time in man-hours required per type of member for one particular design. No allowance was made for supervision. It is also assumed that the men work in teams where full utilization of all personnel is constant. Data for all four designs are included in Table VIII.

Table VII. Estimated Erection Time (Man-hours)

| | Man-hours |
|---|---|
| **(a) Main Shelter (Timber Bulkhead)** | |
| Sills | 8 |
| One Sheathing, Lower Scabs, Lower Spreaders | 8 |
| Lower End Beam (Entrance End) | 1/2 |
| Posts and Stiffeners | 8 |
| Caps | 9 |
| Top Scabs and Spreaders | 5 |
| Upper End Beam (Entrance End) | 1/2 |
| End Posts (Closed End) | 3/4 |
| Sheathing on Bulkhead (Closed End) | 2 |
| Roof Stringers | 25 |
| Side Sheathing, Balance | 35 |
| **(b) Entranceway (With Two Bends)** | |
| Sills | 2 |
| Lower Scabs and Spreaders | 2 |
| Posts | 2 |
| Caps | 1-1/2 |
| Top Scabs and Spreaders | 2 |
| Longitudinal Braces | 1-1/2 |
| Roof Stringers | 1-1/2 |
| Side Sheathing | 7 |
| Bulkhead Sheathing (Main Shelter, Entrance End) | 2 |
| First Two Vertical Beams | 3/4 |
| Braces and Sheathing Between Two Initial Beams | 2 |
| Other Braces and Lowest Sheathing | 1 |
| Two Remaining Beams | 3/4 |
| Remaining Braces and Side Sheathing | 4 |
| Deadmen and Tie Rods | 3/4 |
| Footings and Tie Rod Connecting | 3/4 |
| Hatch Cover Assembly | 1 |
| Fastening of Hatch Cover | 1 |
| Total | 135.25 |

c. **Earthwork**. It is assumed that the excavation will be accomplished before erection of the shelter begins. It is also assumed that the excavation at the bottom of the hole will be oversize 2 feet on each side and each end. Side slopes are predicated at 1 to 1. It is possible that some soils will be sufficiently stable so that essentially vertical walls can be maintained.

Table VIII. Construction Man-hours Required

| Bulk-head Type | No. of Turns in Entrance-way | Erection of Timber (man-hours) | Compacted Backfill | | | Uncompacted Backfill | | | Grand Total (man-hours) |
|---|---|---|---|---|---|---|---|---|---|
| | | | No. of yd$^3$ | Work Rate* (yd$^3$/hr) | Total Man-hours | No. of yd$^3$ | Work Rate* (yd$^3$/hr) | Total Man-hours | |
| Timber | 2 | 134 | 471 | 0.75 | 628 | 1237 | 1.50 | 823 | 1585 |
| Timber | 1 | 122 | 448 | 0.75 | 598 | 1183 | 1.50 | 788 | 1508 |
| Soil | 2 | 161 | 554 | 0.75 | 739 | 1426 | 1.50 | 950 | 1850 |
| Soil | 1 | 149 | 531 | 0.75 | 708 | 1372 | 1.50 | 914 | 1771 |

* Average production rate, 80-percent efficiency, from Reference 13.

However, in unstable soils, even a 1-to-1 slope may be difficult to maintain. Compacted backfill up to the top of the main shelter portion is regarded as necessary. This compaction will tend to increase the mass of the shelter thereby aiding it in resisting loading. It will also cause the overall loading to be more uniform. The succeeding computations employ the average-end area method for determining soil volume. Separate computations are made for the 2-turn and 1-turn entranceways and the timber and soil bulkheads. Data for all four designs are included in Table VIII.

(1) Earthwork (with Horizontal Turn in Entranceway) (Timber Bulkhead) (1-to-1 Slope).

$$\text{Excavation Volume} = \frac{\text{Sum of 2 end areas}}{2} \times \text{depth (for shelter and entrance)}$$

$$= \frac{81.75 \times 44.50 + 67.5 \times 16}{2} \times 14.25$$

$$+ \frac{22.0 \times 43.4 + 8.5 \times 16.4}{2} \times 13.5$$

$$= (3640 + 1080)7.125 + (955 + 140)6.75$$

$$= 4720 \times 7.125 + 1095 \times 6.75 = 33,600 + 7400$$

$$= 41,000 \text{ ft}^3 = 1520 \text{ yd}^3$$

Compaction Volume (total below top of main shelter)

$$= \frac{76.75 \times 34.50 + 67.5 \times 16}{2} \times 9.25 + \frac{17.0 \times 33.4 + 8.5 \times 16.4}{2} \times 8.5$$

$$= 20,250 \text{ ft}^3 = 750 \text{ yd}^3$$

| | | |
|---|---|---|
| Main Shelter Volume | $= 9.21 \times 12.02 \times 63.46$ | $= 7000$ |
| Entrance (Horiz) Volume | $= 6.47 \times 4.35 \times (4.40 + 7.93)$ | $= 347$ |
| Entrance (Vert) Volume | $= 4.60 \times 4.60 \times 13.5$ | $= 286$ |
| Total Shelter Volume | | $= \overline{7633} \text{ ft}^3$ |
| Total Shelter Volume | | $= 283 \text{ yd}^3$ |

Total Backfill      $= 1520 - 283 = 1237 \text{ yd}^3$
Compacted Backfill    $= 750 - 279$ (omit top 5 feet of shaft) $= 471 \text{ yd}^3$

(2) <u>Earthwork (without Horizontal Turn in Entrance-way) (Timber Bulkhead) (1-to-1 Slope)</u>.

Excavation Volume $= \dfrac{\text{Sum of two end areas}}{2} \times$ depth (for shelter and entrance)

$$= \frac{81.75 \times 44.50 + 67.5 \times 16}{2} \times 14.25$$

$$+ \frac{22.0 \times 35.4 + 8.5 \times 8.4}{2} \times 13.5$$

$$= (3640 + 1080)7.125 + (778 + 71)6.75$$

$$= 4720 \times 7.125 + 849 \times 6.75 = 33,600 + 5730$$

$$= 39,330 \text{ ft}^3 = 1457 \text{ yd}^3$$

Compaction Volume (total below top of main shelter)

$$= \frac{76.75 \times 34.50 + 67.5 \times 16}{2} \times 9.25 + \frac{17.0 \times 25.4 + 8.5 \times 8.4}{2} \times 8.5$$

$$= 19,390 \text{ ft}^3 = 718 \text{ yd}^3$$

Structure Volume       = 9.21 x 12.02 x 63.46   =   7000 ft$^3$
Entrance (Horiz) Volume = 6.47 x 4.35 x 4.40    =   125
Entrance (Vert) Volume   = 4.60 x 4.60 x 13.5    =   286
Total Shelter Volume                          =   7411 ft3
Total Shelter Volume                          =   274 yd3

Total Backfill      = 1457 - 274 = 1183 yd$^3$
Compacted Backfill   = 718 - 270 (omit top 5 feet of shaft) = 448 yd$^3$

         (3)   **Earthwork (with Horizontal Turn in Entranceway)**
                  **(Soil Bulkhead) (1-to-1 Slope).**

Excavation Volume (Timber Bulkhead)                 =   1520 yd$^3$

    Add $\dfrac{14.5 \times 44.5 + 14.5 \times 16}{2 \times 27} \times 14.25$              =   232

Excavation Volume (Soil Bulkhead)                    =   1752 yd$^3$

Compaction Volume (Timber Bulkhead)               =   750 yd$^3$

    Add $\dfrac{14.5 \times 34.5 + 14.5 \times 16}{2 \times 27} \times 9.25$              =   126

    Add $\dfrac{(4 + 7)(11 + 6.5)7.75}{2 \times 27}$ (Actual Bulkheads)     =   28

Compaction Volume (Soil Bulkhead)                   =   904 yd$^3$

Total Shelter Volume (Timber Bulkhead)            =   283 yd$^3$

     Add (9.21 x 12.02 x 17.58 x 1/27 - 6.47 x 4.35
         x 1.04 x 1/27)                            =   71
                                                 354 yd3

Total Backfill = 1752 - 354 + 28 = 1426 yd$^3$
Compacted Backfill = 904 - 350 (omit top 5 feet of shaft) = 554 yd$^3$

         (4)   **Earthwork (without Horizontal Turn in Entrance-**
                  **way) (Soil Bulkhead) (1-to-1 Slope).**

Excavation Volume (Timber Bulkhead)                 =   1457 yd$^3$

    Add $\dfrac{14.5 \times 44.5 + 14.5 \times 16}{2 \times 27} \times 14.25$              =   232

Excavation Volume (Soil Bulkhead)                    =   1689 yd$^3$

Compaction Volume (Timber Bulkhead)               =   718 yd$^3$

    Add $\dfrac{14.5 \times 34.5 + 14.5}{2 \times 27} \times 9.25$                 =   126

    Add $\dfrac{(4 + 7)(11 + 6.5)7.75}{2 \times 27}$ (Actual Bulkheads)     =   28

Compaction Volume (Soil Bulkhead) $\qquad$ = 872 yd$^3$

Total Shelter Volume (Timber Bulkhead) = 274 yd$^3$
    Add 9.21 x 12.02 x 17.58 x 1/27 - 6.47 x 4.35
        x 1.04 x 1/27 = 71
Total Shelter Volume (Soil Bulkhead) = 345 yd$^3$

Total Backfill = 1689 - 345 + 28 = 1372 yd$^3$
Compacted Backfill = 872 - 341 (omit top 5 feet of shaft) = 531 yd$^3$
Uncompacted Backfill = 854 - 332 = 522 yd$^3$

    7.   Construction Procedure.

        a.  Main Shelter. Prepare level soil bed for sills by using chalkline and carpenter's levels. Drive drift pins (1/2 by 16 inches) into each end of each sill. Drift pin holes should be pre-bored 1/8 inch smaller than pin diameter. Place sills end-to-end, each side, with head of drift pin on bottom and exposed point on top. Fasten sills together by nailing bottom scabs and bottom piece of side sheathing into place. Fasten bottom spreaders into position by toenailing (use long nails, 60d or larger). Place lower end beam (entrance end) into position and connect with drift pins.

        Pre-drill drift pin holes (two required) 1/8 inch oversize in one end of each post and 1/8 inch undersize in other end of each post. Alternate installing posts on each side. Place posts onto drift pins in sills (use end with oversize holes to obviate driving posts). Nail each post into position by installing diagonal stiffeners and completing nailing of scab. Pre-drill 1/8-inch undersize drift pin holes in each end of cap. Place caps on posts, and connect with drift pins. Nail in top scabs, and toenail in top spreaders. Place and secure two end posts and two end beams with drift pins. Drift pin holes have to be pre-bored in beams and end caps. Place and secure sheathing (use long spikes) on closed end bulkhead. Sheathing on entrance bulkhead should not be placed until entranceway has been completed. Place soil floor in a compacted manner level with top of sills.

        Place and secure with long spikes all roof stringers. Nail on side sheathing, alternating the joints from bottom to top. The foregoing procedure will vary somewhat when soil bulkheads are employed. The end roof stringers would be placed after the soil bulkheads are placed.

        b.  Entranceway (Horizontal Portion). Erect this portion of structure in similar manner as main shelter. Install longitudinal braces right after top spreaders are installed. Posts adjacent to

each other should be singly bolted together using pre-drilled holes, 1/8 inch oversize, and nuts and washers. After stringers and sheathing are installed, the entrance-end bulkhead sheathing of main shelter (timber bulkhead) should be fastened into place with long spikes. Length of this portion of entranceway would be reduced if only one bend (vertical) is required.

c. <u>Entranceway (Vertical Portion)</u>. Place two vertical beams, one at a time, against two end posts (horizontal portion) and bolt into position with single bolts. Bolt holes should be pre-drilled, 1/8 inch oversize, through beam and post. Use washer and nut fastenings for bolts. Toenail three braces between these beams into position. Nail into position sheathing connecting these beams. Toenail other bottom braces, and nail bottom side sheathing to erected beams. Erect two remaining beams, one at a time, by fastening to bottom sheathing and brace and toenailing remaining middle and top braces. Add additional side sheathing to maintain erect position. Install three remaining braces by toenailing. Compact soil flooring level with top of sills in horizontal entranceway. Complete the nailing of side sheathing.

Place deadmen into position during backfill. Fasten tie rods to deadmen. Place footings during backfill and connect to tie rods. Assemble two-part hatch cover. Attach cover to footings.

d. <u>Compacted Backfill</u>. All backfill placed below a plane level with the top of the main shelter should be compacted. Compaction should be in 6-inch lifts maximum and should be kept within one lift level all around the structure. Compaction may be accomplished by foot stamping or with hand-tampers. Adding water to very dry soils may be necessary for adequate compaction.

e. <u>Backfill</u>. All backfill other than compacted backfill should be placed in an approximately level manner to obviate air pockets. The deadmen tie rods for the hatch cover have to be placed during placement of backfill. Final stages of the backfill can only be accomplished after entranceway hatch cover is in place.

f. <u>Hatch Cover</u>. The cover stringers should be multiple nailed to beam so as to approach a fixed end connection. Because of heavy weight of cover stringers, it is necessary that the hatch cover be in two equal parts. The siding should be double nailed to end of beam to maintain rigidity. After backfill is mostly complete, the footings should be set in place and connected by tie rods to the deadmen. The hatch cover (two parts) should then be hinged to one footing and fasteners provided for connection to the other footing. After the hatch cover has been completed, the balance of the backfill should then be performed.

REFERENCES

1. Department of the Army, "The Effects of Nuclear Weapons," Pamphlet No. 39-3 (Washington, D. C.: U. S. Government Printing Office, April 1962).

2. National Lumber Manufacturers Association, "National Design Specifications for Stress-Grade Lumber and Its Fastenings" (Washington, D. C.: National Lumber Manufacturers Association, 1960).

3. Newmark, N. M., J. W. Briscoe, and J. L. Mcrritt, Analysis and Design of Flexible Underground Structures, Volume I (Urbana, Illinois: Contract DA-22-079 Eng-225, 1 May 1960).

4. Department of the Army, Field Fortifications, FM 5-15 (Washington, D. C.: U. S. Government Printing Office, October 1959).

5. Marshall, R. G., and J. E. Pleasants, "Hasty Personnel Shelter for Protection from Atomic Effects," Report 1571-TR (Ft. Belvoir, Va.: U. S. Army Engineer Research and Development Laboratories, 9 March 1959).

6. Department of the Army, Engineer Troop Protective Construction (Nuclear Warfare), TM 5-311 (Washington, D. C.: U. S. Government Printing Office, August 1961).

7. Department of Agriculture, Wood Handbook, Agriculture Handbook No. 72 (Washington, D. C.: U. S. Government Printing Office, 1955).

8. National Lumber Manufacturers Association, Wood Structural Design Data, Vol. I (Washington, D. C.: National Lumber Manufacturers Association, 1957).

9. Department of the Navy, Bureau of Yards and Docks, Studies in: Atomic Defense Engineering, NAVDOCKS P-290.2 (Washington, D. C.: U. S. Government Printing Office, November 1961).

10. Jacoby, H. S., and R. P. Davis, Foundations of Bridges and Buildings (New York, N. Y.: McGraw-Hill Book Co., Inc., 1941).

11. Department of the Army, Design of Structures to Resist the Effects of Atomic Weapons, Corps of Engineers Manuals EM 1110-345-413 - EM 1110-345-421 (Washington, D. C.: U. S. Government Printing Office, 1957-1961).

36

12. Strope, W. E., et al, Specifications and Costs of a Standardized Series of Fallout Shelters, USNRDL-TR-366 (San Francisco, Calif.: U. S. Naval Radiological Defense Laboratory, 6 October 1959)..

13. Dallavia, L., Estimating Production and Construction Costs (Houston, Texas: Dallavia Co., 1954).

D

C

B

A

63'-5½"

SPREADERS

4-7¼"

4'-3"

4'-3"

3'-0"

7-8½"

4-4¼"

5-9½"

6"x6"

5'-0"

4-1⅛"

9½"

3'-0"

3'-10"

K

10'-0"

L

**PLAN**
SCALE ½"=1'-0"

END SHEATHING
8"x5"x8'-6", 12 REQUIRED

END SHEATHING 16"x5"x5'-8"
I REQUIRED

END BEAM

STRINGERS 8"x8"x11'-0"
100 REQUIRED

9½"

STRINGERS
12"x3"x3'-11"
6 REQUIRED

7½"

SPREADERS

SCABS

POST

7'-0"

2x4 BRACING
SEE DETAIL ON
SHEET 2.

SPREADERS

BRACE
6"x2½"x3'-0"
2 REQUIRED

18"   18"

BRACE

5'-0"

END SHEATHING 10"x6"x11'-0
II REQUIRED

2x4 BRACING 6 SLANTED
DIAGONALLY AS SHOWN
EACH SIDE

END POST
10x10"x8-7
2 REQUIRED

2x4 BRACING, 6 EACH SIDE
SLANTED DIAGONALLY
AS SHOWN

9½"

SILL

7½"  POST
10"x6"x5'-0"
I REQ'D.

END BEAM
2 REQUIRED
10"x10"x11'-7

SPREADER

SCAB 8"x3"x15'
2 REQUIRED PER POST

16¼"    13 @ 5'-0" = 60'-0" POSTS, SPREADERS + SCABS    15¼"

**SECTION K-K**
SCALE ½"=1'-0"

END SHEATHING
8"x5"x8'-6", 12 REQUIRED

END SHEATHING 6"x5"x5'-8"
1 REQUIRED

END BEAM

STRINGERS
12"x3"x3'-11"
6 REQUIRED

SPREADERS

STRINGERS 12"x3"x3'-11"
8 REQUIRED

CAP 6"x8"x7'-8½"
1 REQUIRED PER SIDE

BRACE
SIDE SHEATHING

BRACE
6"x2½"x3'-0"
2 REQUIRED

POST 12"x6"x5'-0"
1 REQUIRED

BRACE 6"x2½"x2'-7"
2 REQUIRED

POST
10"x6"x5'-0"
1 REQ'D.

SPREADER

SILL 6"x8"x7-8½"
1 REQUIRED PER SIDE

END BEAM
2 REQUIRED
10"x10"x11'-7"

SCAB 8"x3"x15"
2 REQUIRED PER POST

| APPLICATION | | UNLESS OTHERWISE SPECIFIED | | U. S. ARMY ENGINEER RESEARCH AND DEVELOPMENT LABORATORIES CORPS OF ENGINEERS FORT BELVOIR, VA |
|---|---|---|---|---|
| NEXT ASSY | USED ON | DIMENSIONS ARE IN INCHES TOLERANCES ON FRACTIONS DECIMALS ANGLES | | CIVILIAN GROUP SHELTER |
| | | BREAK SHARP EDGES TO ALL FILLETS TO | | TIMBER |
| | | MATERIAL | | |

CODE IDENT. NO. 97403

SIZE D  13208E5646

SCALE: AS SHOWN   SHEET 1 OF 4

D

CAP, 10"x10"x 5'-6¾", 1 REQUIRED
PER SIDE, CLOSED END

STRINGERS

CAP, 10"x10"x5'-0", 1 REQUIRED
PER SIDE, MIDDLE UNITS

STRING

CAP 10"x10"x5'-5¾"
1 REQUIRED PER SIDE,
ENTRANCE END

SPREADER

SCAB

SCAB

SHEATHING CUT TO
LENGTHS AS SHOWN
IN ELEVATION

SHEATHING CUT TO
LENGTHS AS SHOWN IN
ELEVATION

POST, 12"x10"x7'-0"
13 REQUIRED
PER SIDE

7'-0"

POST

POST

10'-0"

SCAB

SCAB

C

SPREADER 6"x6"x9'-4¾"
2 REQUIRED PER PAIR OF POST

SILL 10"x10"x5'-0", 1 REQUIRED
PER SIDE, MIDDLE UNITS

SPREAD

SILL 10"x10"x 5'-6¾"
1 REQUIRED PER
SIDE, CLOSED END

SCAB 12"x4"x20", 2 REQUIRED
PER POST

SILL 10"x10"x 5'-5¾"
1 REQ'D PER SIDE
ENTRANCE END

**SECTION L·L**
SCALE 3/4"·1'-0"

**SECTION M·M**
SCALE 3/4"·1'-0"

B

5'-0"

2¾"

8'-0 15/16"

1¾"

2x4 BRACING

7'-0"

2¾"

STRINGERS 12"x3"x 3'-11", 6 REQUIRED

SPREADER

CAP 6"x6"x5'-9½" 1 REQUIRED
PER SIDE

SCAB

SIDE SHEATHING 10"x3"x5'-7"
8 REQUIRED

18"

POST 10"x6"x5'-0" 2 REQUIRED
PER SIDE

SCAB 10"x2½"x15", 2 REQUIRED
PER POST

SPREADER 4"x4"x2'-7¾", 2
REQUIRED PER PAIR OF
POSTS

A

SILL 6"x8"x 5'-9½" 1 REQUIRED
PER SIDE

**BRACING DETAIL**
SCALE 3/4"·1'-0"

**SECTION A·A**
SCALE 3/4"·1'-0"

CAP, 10"x10"x5'-0" 10 REQUIRED
PER SIDE, MIDDLE UNITS

CAP 10"x10"x5'-5¾"
1 REQUIRED PER SIDE
ENTRANCE END

STRINGERS

END BEAM

SHEATHING CUT TO
LENGTHS AS SHOWN IN
ELEVATION

END SHEATHING 8"x5"x8'-6"
12 REQUIRED

POST

SCAB

10"x10"x5'-0" 10 REQUIRED
SIDE, MIDDLE UNITS

END BEAM

SILL 10"x10"x5'-5¾"
1 REQ'D PER SIDE
ENTRANCE END

SPREADER

SECTION M-M
SCALE 3/4"=1'-0"

STRINGERS 12"x3"x3'-11", 6 REQUIRED

SPREADER

CAP 6"x8"x5'-9½" 1 REQUIRED
PER SIDE

SCAB

SIDE SHEATHING 10"x3"x5'-7¾"
8 REQUIRED

18

POST 10"x6"x5'-0" 2 REQUIRED
PER SIDE

x2½"x15", 2 REQUIRED
T

4"x4"x2-7¾", 2
PER PAIR OF

SILL 6"x8"x5'-9½" 1 REQUIRED
PER SIDE

SECTION A-A
SCALE 3/4"=1'-0"

ELEVATION
OF SIDING DETAIL
SCALE 1/2"=1'-0"

10'-0"

3'-9¼"

8'-3¼"

11'-3¼"       5'-0"

D

STRINGERS 8 x 3 x 5'-4¼"
8 REQUIRED

SILL 12 x 4 x 5'-6"
2 REQUIRED

BEAM 3 x 4 x 5

SIDING

SHEATHING 12 x 2½ x 4'-3"

SHEATHING 12 x 2½ x 4'-3"

BRACE
4 REQ

C

CAP 6 x 8 x 7'-8½"

BRACE 6 x 2½ x 2'-11"
2 REQUIRED

POST

POST 6 x 6 x 5'-0"

BRACE
4 REQ

SIDE SHEATHING
10 x 2½ x 3'-9½", 8 REQUIRED

5'-0"

12'-2⅝"

SHEATHING 10 x 2½ x 3'-10¼"
8 REQUIRED, & 40 10 x 2½ x 7'-5½"
ALTERNATE LONG & SHORT
SHEATHING BOTTOM TO TOP

POST
4 REQ

BRACE 6 x 2½ x 3'-0", 2
REQUIRED

B

BRACE

BRACE
4 REQ

SILL 6 x 8 x 7'-8½"

SCAB

SCAB 6 x 3 x 15"

SHEATHING
13 REQUIRED
SIDES

SPREADER 4 x 4 x 2'-6¾"

### SECTION D-D
SCALE 3/4"·1'-0"

SIDING

STRINGERS

SIDING 3 x
2 REQUIRED

SILL

BEAM

¼" DIA. STEEL

3"

3"

A

DEAD MAN 6 x 4 x 5'-6"
2 TYP.

### SECTION H-H
SCALE 3/4"·1'-0"

3     2     1

| SYM | ZONE | DESCRIPTION | DATE | APPROVAL |
|---|---|---|---|---|
| | | REVISIONS | | |

D

SILL 12x4x5'-6"
2 REQUIRED

STRINGERS 8x3x5'-4¼"
8 REQUIRED

BEAM 3x4x5'-0"

SIDING

H

¼x4'3"

BRACE 4x4x3'-0"
4 REQUIRED

30°

6'-5⅟₁₆"

—3'-0"

¼ DIA STEEL ROD, 4 TYP

H

DEAD MAN 6x4x5'-6"
2 TYP

SHEATHING 12x2½x4'-3"
6 REQUIRED

STRINGERS 12x3x3'-11"
8 REQUIRED

C

2½x3-10¼"
10x2½x7-5½"
LONG & SHORT
BOTTOM TO TOP

BRACE 6x5x3'-0"
4 REQUIRED

SCAB 6x3x15"
2 PER POST

CAP

SCAB 6"x3"x15"
2 PER POST, EACH SIDE

POST 10"x6"x5'-0" THIS SIDE, 1 REQUIRED
POST 6"x6"x5'-0" OTHER SIDE, 1 REQUIRED

6'-5⅟₁₆"

POST 8x8x12'-2⅝"
4 REQUIRED

POST 6x6x5'-0"
1 PER SIDE

POST 12x6x5'-0"

SHEATHING 10x3x9-8½"
8 REQUIRED

SPREADER
5x6x2'-6¼"
2 REQUIRED

BRACE

BRACE 4x4x3'-0"
4 REQUIRED

BRACE

SPREADER 4"x4"x2'-6¾"
2 PER PAIR OF POSTS.

SHEATHING 12x2½x4'-3"
13 REQUIRED EACH OF THREE
SIDES

SPREADER 4"x4"x2'-6¾"
2 PER PAIR OF POSTS

SCAB 8x3x15"
2 PER POST

SILL

SECTION G-G
SCALE 3/4"=1'-0"

B

NG

SIDING 3x4x5'-4¼"
2 REQUIRED

STRINGERS

2 ¼ DIA STEEL ROD, 4 TYP

SILL   BEAM

3"    3"

DEAD MAN 6x4x5'-6"
2 TYP

SECTION H-H
SCALE 3/4"=1'-0"

A

| APPLICATION | | UNLESS OTHERWISE SPECIFIED | |
|---|---|---|---|
| NEXT ASSY | USED ON | DIMENSIONS ARE IN INCHES TOLERANCES ON FRACTIONS DECIMALS ANGLES | |
| | | BREAK SHARP EDGES TO ALL FILLETS TO | |
| | | MATERIAL | |

RECOMMENDED: S.K.Royudon
SUBMITTED: Edward Freeland
REVIEWED FOR PRODUCTION:

| DRAWN BY | TRACED BY | CHECKED BY |
|---|---|---|
| | | |
| APPROVED: | | |
| CONTRACTOR: | | |

U. S. ARMY
ENGINEER RESEARCH AND DEVELOPMENT
LABORATORIES   CORPS OF ENGINEERS
FORT BELVOIR, VA

CIVILIAN GROUP SHELTER

TIMBER

CODE IDENT. NO. 97403   SIZE D   13208E5646

SCALE: AS SHOWN    SHEET 5 OF

2

STRINGERS 8" x 8" x 11' 0"
16 ADDITIONAL REQ'D

SPREADER 6" x 6", 9 - 4 ¼"
4 ADDITIONAL REQ'D

SCAB 12" x 4" x 20"
8 ADDITIONAL REQ'D

CAP 10 x 10 x 5' x 5 ¾"

POST 12", 10", 7' - 0"
4 ADDITIONAL REQ'D

CAP 6" x 8" x 5' - 9½"
2 ADDITIONAL REQ'D

CAP 10" x 10" x 5' - 0"
4 ADDITIONAL REQ'D

POST 10" x 6" x 5' - 0"
4 ADDITIONAL REQ'D

STRINGERS 12", 3", 3' - 11"

SCAB 10", 2½" x 15"
4 ADDITIONAL REQ'D

SPREADER 4" x 4", 2' - 7 ¾"
4 ADDITIONAL REQ'D

earth line

SILL 10" x 10" x 5' - 0"
4 ADDITIONAL REQ'D

SPREADER 4" x 4" x 2' - 7 ¾"
4 ADDITIONAL REQ'D

SCAB 10" x 2½" x 15"
8 ADDITIONAL REQ'D

SILL 6" x 8" x 5' - 9½"
2 ADDITIONAL REQ'D

SILL 10" x 10", 5' - 5 ¾"

### SECTION N-N
SCALE ½" = 1' 0"

CAP 10" x 10" x 5 - 6 ¾"

STRINGERS 8" x 8", 11' - 0"
16 ADDITIONAL REQ'D

CAP 10", 10" x 5' - 0"
4 ADDITIONAL REQ'D

POST 12" x 10" x 7' - 0"
4 ADDITIONAL REQ'D

SCAB 12" x 4" x 20"
8 ADDITIONAL REQ

SPREADERS 6" x 6", 9' - 4 ¾"
4 ADDITIONAL REQ'D

earth line

SILL 10" x 10" x 5 - 6 ¾"

SILL 10" x 10" x 5' - 0"
4 ADDITIONAL REQ

### SECTION P-P
SCALE ½" = 1' 0"

ALT

ENTRANCE

P

D

`7-0`
`REQD`
`REQD`

`POST 10 x 5 x 5 - 0`
`4 ADDITIONAL REQD`

`SCAB 0 x 2½ x 15`
`4 ADDITIONAL REQD`

`SPREADER 4 x 4 x 2 - 7¾`
`4 ADDITIONAL REQD`

C

NOTE

1 ALTERNATE ENTRANCE CONSTRUCTION IDENTICAL
TO ENTRANCE ON SHEET D     EXCEPT AS
SHOWN.
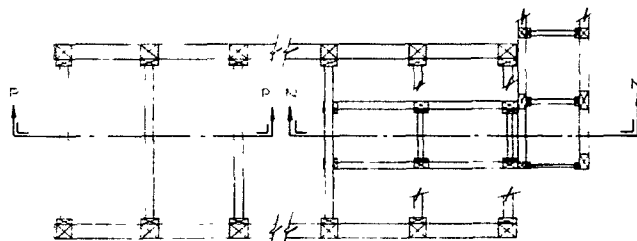


ALTERNATE PLAN
OF
ENTRANCE AND BULKHEAD
SCALE ¼ = 1 - 0

`CAP 10 x 10 x 5 - 0`
`4 ADDITIONAL REQD`

B

`SLAB 12 x 4 x 20`
`ADDITIONAL REQ`

`earth line`

`5 - 0`
`AL REQ`

A

APPENDIX C                                    <u>Exhibit 1</u>

METAL STRUCTURES

by

Malcolm K. Dale

1.  <u>General</u>.  The basic design considerations for the metal
shelter are similar to those guiding the designs in the other ap-
pendices to the report.  The shelter is designed for occupancy by
60 persons and to resist a static pressure of 23.5 psi.  A soil
cover of 5-foot depth accounts for 3.5 psi of this pressure with
the remaining 20 psi as resistance against the "dynamic-static"
pressure to be anticipated at a certain distance from a multi-
megaton burst.  The shelter with 5-foot soil cover should be safe
from fallout dangers and from incident nuclear and thermal radia-
tion within a zone where blast pressure does not exceed 20 psi.

2.  <u>Design Type</u>.  Corrugated metal offers considerably more
strength than non-corrugated metal for a given size and weight.  In
addition, considerable engineering data is available on corrugated
metal (in particular, steel) as used in culvert construction.  Cor-
rugated steel culvert has been proved through many years of usage,
buried in all kinds of soil, to offer tremendous resistance to loads
and to offer excellent corrosion resistance.  Culvert is available
commercially in two size corrugations, a 1/2-inch-deep by 2-2/3-inch
pitch corrugation and a 2-inch-deep by 6-inch pitch corrugation.
The larger corrugation is called sectional plate and is considerably
stronger with a higher section modulus.  Steel is available commer-
cially in both size corrugations, while aluminum is available only
in the small corrugation.  The small aluminum culvert has received
sufficient testing to be considered for use.  Aluminum sectional
plate (large corrugations) is presently in the design and test stage
of development by some of the large aluminum companies.  Test in-
stallations for determination of load ratings are planned for the
fall of 1962, with tentative plans to make the sectional plate com-
mercially available sometime during the summer of 1963.  Aluminum
offers certain distinct advantages over steel for this application.
Its density and, therefore, weight is approximately one-third that
of steel (0.1 pound per cubic inch vs 0.281 pound per cubic inch).
In addition, aluminum is very workable and provides excellent corro-
sion resistance.  Structural aluminum offers sufficient mechanical
properties to make it practically as strong as steel for most appli-
cations, which utilizes the weight difference.  Larger size sectional

plate sections could be used; this would reduce the number of seams and, therefore, bolting times. Aluminum is costly, and no data is available, as previously mentioned, on aluminum sectional plate in the common culvert shapes (outlined below). However, the section modulus for strength design of flat corrugated metal plate is dependent only on corrugation size and not on material. Therefore, knowing the mechanical properties of aluminum and the section modulus, the strength required for various loadings of flat corrugated sheets can be calculated in bending and shear with reasonable accuracy. Based on the above facts, it was decided to use steel sectional plate in commercially available standard culvert shapes for shelter and main entranceway bodies. Aluminum beams and sectional plate would be used in the shelter bulkheads and a combination of aluminum and steel in the entranceway bulkhead. Galvanic action may take place at the joining points (contact points) between two dissimilar metals such as aluminum and steel. This could mean relatively rapid joint corrosion. Therefore, it is recommended that all joints (both aluminum and steel members) be painted with a zinc-yellow primer or rubber base paint prior to assembly. These paints are good corrosion prohibiters on both galvanized steel and aluminum.

Corrugated sectional plate steel culvert is available commercially in the following cross sections:

a.  Pipe

b.  Pipe arch

c.  Arch

d.  Underpass

The pipe (circular) cross section will withstand the greatest loading for a given gage and cross-sectional area. However, it offers a minimum of headroom for a reasonable floor space. The arch shape offers extremely flat floor space but presents foundation and waterproofing problems (if waterproofing is necessary). The underpass shape gives excellent headroom and a flat floor but requires considerably heavier gages for the same loading, making its use prohibitive in the shelter cross-section area range desired. The pipe arch seems to represent a good compromise of all the above shapes and was, therefore, chosen. It does not require special foundations or heavy gages, but it does give reasonably good headroom and fairly flat floor space. The construction of a flat wooden floor if desired would be quite simple.

The underpass shape, which gives the maximum headroom and a flat floor for a given cross-sectional area, was used in the entranceway since it did not require an extremely heavy gage for the cross-sectional area and headroom desired.

3.   Design Discussion.  Bill of materials is shown in Table IX.  The lengths of the steel sectional plate used in shelter and entranceway construction are limited to 2 and 4 feet rather than to the standard 6 and 8 feet.  This was necessary in order to reduce the weight to an amount more capable of being handled.  The heaviest individual shelter sectional plate section weighs 194 pounds and can be handled by 3 to 5 men (40 to 65 pounds per man).  The heaviest individual entranceway sectional plate section weighs 198 pounds and can be similarly handled.

The total weight of the rear shelter bulkhead is approximately 50 pounds and the front bulkhead approximately 350 pounds including connections but excluding deadmen.  The heaviest aluminum beam weighs approximately 40 pounds, and the heaviest corrugated aluminum sheet weighs approximately 70 to 90 pounds.  Therefore, all bulkhead components can be manhandled.

The hatchway consists of two sections of corrugated aluminum pipe (some engineering data is available on small-diameter aluminum culvert pipe, mentioned previously, enabling it to be used here).  The lower section, attaching directly to the entranceway and consisting of a half cylinder, weighs approximately 50 pounds.  The upper section weighs approximately 80 pounds.

The total cover weight is approximately 150 pounds, and its heaviest component weighs 29 pounds.  Therefore, the cover can be assembled and handled by two men.  The hinging and locking connections need withstand only forces from negative pressures which are extremely low (2 to 3 psi) creating a negative force on the cover of about 6,000 pounds.  Practically any simple hinging apparatus will suffice so the detail design is not given.  All that is necessary is a bolt or bolts of 0.12 inch$^2$ total cross-section area or more in tension or 0.6 inch$^2$ in shear.

To simplify assembly, a sectional plate numbering system has been developed for both shelter and entranceway.  In the same manner, a system may be used in bulkhead assembly, if proved necessary.

The excavation it is assumed will be made as safe as possible against cave-ins considering the inexperienced person to be working within it since they may not recognize a dangerous situation.

Table IX.  Bill of Materials, Drawing No. 13208E5648

| Major Assembly | Sub-Assembly | Item Description | No. Req'd | Sheet No. | Material |
|---|---|---|---|---|---|
| Shelter | Pipe-Arch | Corner<br>Sectional Plate; 9 Pi Wide, 4 ft Long | 36 | 2 | Steel |
| Shelter | Pipe-Arch | Invert<br>Sectional Plate; 15 Pi Wide, 4 ft Long | 18 | 2 | Steel |
| Shelter | Pipe-Arch | Side<br>Sectional Plate; 18 Pi Wide, 4 ft Long | 38 | 2 | Steel |
| Shelter | Pipe-Arch | Top<br>Sectional Plate; 21 Pi Wide, 4 ft Long | 18 | 2 | Steel |
| Shelter | Pipe-Arch | Corner<br>Sectional Plate; 9 Pi Wide, 2 ft Long | 4 | 2 | Steel |
| Shelter | Pipe-Arch | Invert<br>Sectional Plate; 15 Pi Wide, 2 ft Long | 2 | 2 | Steel |
| Shelter | Pipe-Arch | Top<br>Sectional Plate; 21 Pi Wide, 2 ft Long | 2 | 2 | Steel |
| Shelter | Pipe-Arch | Invert<br>Sectional Plate; 18 Pi Wide, 4 ft Long | 19 | 2 | Steel |
| Entranceway | Underpass<br>Section No. 1 | Corner<br>Sectional Plate; 9 Pi Wide, 2 ft Long | 2 | 3 | Steel |
| Entranceway | Underpass<br>Section No. 1 | Corner<br>Sectional Plate; 9 Pi Wide, 4 ft Long | 3 | 3 | Steel |

Table IX (cont'd)

| Major Assembly | Sub-Assembly | Item Description | No. Req'd | Sheet No. | Material |
|---|---|---|---|---|---|
| Entranceway | Underpass Section No. 1 | Invert Sectional Plate; 9 Pi Wide, 4 ft Long | 2 | 3 | Steel |
| Entranceway | Underpass Section No. 1 | Invert Sectional Plate; 9 Pi Wide, Special Varying Length | 1 | 3 | Steel |
| Entranceway | Underpass Section No. 1 | Corner Sectional Plate; 9 Pi Wide, Special Varying Length | 1 | 3 | Steel |
| Entranceway | Underpass Section No. 1 | Corner Sectional Plate; 9 Pi Wide, Special Varying Length | 1 | 3 | Steel |
| Entranceway | Underpass Section No. 1 | Side Sectional Plate; 15 Pi Wide, 4 ft Long | 3 | 3 | Steel |
| Entranceway | Underpass Section No. 1 | Side Sectional Plate; 15 Pi Wide, Special Varying Length | 1 | 3 | Steel |
| Entranceway | Underpass Section No. 1 | Side Sectional Plate; 15 Pi Wide, Special Varying Length | 1 | 3 | Steel |
| Entranceway | Underpass Section No. 1 | Top Sectional Plate; 21 Pi Wide, 2 ft Long | 1 | 3 | Steel |

Table IX (cont'd)

| Major Assembly | Sub-Assembly | Item Description | No. Req'd | Sheet No. | Material |
|---|---|---|---|---|---|
| Entranceway | Underpass Section No. 1 | Top Sectional Plate; 21 Pl Wide, 4 ft Long | 1 | 3 | Steel |
| Entranceway | Underpass Section No. 1 | Top Sectional Plate, Varying Width and Length | 1 | 3 | Steel |
| Entranceway | Underpass Section No. 2 | Invert Sectional Plate; 9 Pl Wide, 4 ft Length | 2 | 3 | Steel |
| Entranceway | Underpass Section No. 2 | Invert Sectional Plate; 9 Pl Wide, Special Varying Length | 1 | 3 | Steel |
| Entranceway | Underpass Section No. 2 | Corner Sectional Plate; 9 Pl Wide, 2 ft Length | 2 | 3 | Steel |
| Entranceway | Underpass Section No. 2 | Corner Sectional Plate; 9 Pl Wide, 4 ft Length | 3 | 3 | Steel |
| Entranceway | Underpass Section No. 2 | Corner Sectional Plate; 9 Pl Wide, Special Varying Length | 1 | 3 | Steel |
| Entranceway | Underpass Section No. 2 | Corner Sectional Plate; 9 Pl Wide, Special Varying Length | 1 | 3 | Steel |
| Entranceway | Underpass Section No. 2 | Side Sectional Plate; 15 Pl Wide, 4 ft Length | 3 | 3 | Steel |

Table IX (cont'd)

| Major Assembly | Sub-Assembly | Item Description | No. Req'd | Sheet No. | Material |
|---|---|---|---|---|---|
| Entranceway | Underpass Section No. 2 | Side Sectional Plate; 15 Pi Wide, Special Varying Length | 1 | 3 | Steel |
| Entranceway | Underpass Section No. 2 | Side Sectional Plate; 15 Pi Wide, Special Varying Length | 1 | 3 | Steel |
| Entranceway | Underpass Section No. 2 | Top Sectional Plate; 21 Pi Wide, 4 ft Long | 1 | 3 | Steel |
| Entranceway | Underpass Section No. 2 | Top Sectional Plate; 21 Pi Wide, 2 ft Long | 1 | 3 | Steel |
| Entranceway | Underpass Section No. 2 | Top Sectional Plate; 21 Pi Wide, Special Varying Length | 1 | 3 | Steel |
| Entranceway | Hatchway | Lower Section, Half Cylinder | 1 | 4 | Aluminum |
| Entranceway | Hatchway | Upper Section, Full Cylinder | 1 | 4 | Aluminum |
| Entranceway | Hatchway | Connecting Band | 1 | 4 | Aluminum |
| Shelter | Front & Rear Bulkhead | 8 WF 5.90 Beam     Ft Long | 5 | 7 | Aluminum |
| Shelter | Front & Rear Bulkhead | 4  2.16 Beam     Ft Long | 7 | 7 | Aluminum |

Table IX (cont'd)

| Major Assembly | Sub-Assembly | Item Description | No. Req'd | Sheet No. | Material |
|---|---|---|---|---|---|
| Shelter | Front & Rear Bulkhead | 4 I 2.64 Beam    Ft Long | 9 | 7 | Aluminum |
| Shelter | Front & Rear Bulkhead | Left Side Sectional Plate, Special Shape | 2 | 5&6 | Aluminum |
| Shelter | Front & Rear Bulkhead | Right Side Sectional Plate, Special Shape | 2 | 5&6 | Aluminum |
| Shelter | Rear Bulkhead | Center Sectional Plate, Special Shape | 1 | 6 | Aluminum |
| Shelter | Front Bulkhead | Top, Center Sectional Plate, Special Shape | 1 | 5 | Aluminum |
| Entranceway | Bulkhead | 3  4.1  Beam, Special Bend | 4 | 8 | Steel |
| Entranceway | Bulkhead | 4 I 7.7 Beam, by 7 Ft Long | 2 | 8 | Steel |
| Entranceway | Bulkhead | 10 WF 7.51 Beam, by 13 Ft Long | 2 | 8 | Aluminum |
| Entranceway | Bulkhead | 1-1/2 x 2 x 1/4  Beam, 6'-6" Long | 4 | 8 | Steel |
| Entranceway | Bulkhead | 3/4 in. Diameter by 32" Long Rod, th'd ends | 8 | 8 | Steel |
| Entranceway | Bulkhead | 6'-6" Wide by 15-1/4" Lg Sectional Plate | 2 | 8 | Aluminum |
| Entranceway | Bulkhead | 3  6 Beam, by 38 in. Long | 1 | 8 | Steel |

Table IX (cont'd)

| Major Assembly | Sub-Assembly | Item Description | No. Req'd | Sheet No. | Material |
|---|---|---|---|---|---|
| Shelter | Peripheral & Longitudinal Seams, Pipe-Arch | Bolts, 3/4-10 NG-2A x As Required | 2921 | 2 | Hi-Str Steel |
| Entranceway | Peripheral & Longitudinal Seams, Underpass | Bolts, 3/4-10 NG-2A x As Required | 944 | 3 | Hi-Str Steel |
| Entranceway | Hatchway & Bulkhead | Bolts, 1/2 Diam. Bolts, Length as Required | 48 | 8 | Hi-Str Steel |
| Entranceway | Hatchway | Connecting Band, 36" Diam. x 12" Long | 1 | 8 | Aluminum |
| Entranceway | Bulkhead | 5/8 in. Diam. Bolts x 3 in. Long | 18 | 8 | Hi-Str Steel |
| Shelter | Bulkheads | 5/8 in. Diameter Bolts x 2 in. Long | 40 | 5&6 | Hi-Str Steel |
| Shelter | Bulkheads | 7/8 in. Diam. Bolts x 2-1/2 in. Long | 88 | 5&6 | Hi-Str Steel |
| Shelter | Bulkheads | 3 x 3 x 2 x 3/8 in. Connect Bracket | 24 | 5&6 | Aluminum |
| Shelter | Bulkheads | 3 x 3 x 8 x 3/8 in. Connect Bracket | 12 | 5&6 | Aluminum |
| Shelter | Bulkheads | 3 x 3 x 2-3/4 x 3/8 in. Connect Bracket | 6 | 5&6 | Aluminum |
| Shelter | Bulkheads | 8 x 5-1/4 x 3/8 in. End Plate | 12 | 5&6 | Aluminum |

Table IX (cont'd)

| Major Assembly | Sub-Assembly | Item Description | No. Req'd | Sheet No. | Material |
|---|---|---|---|---|---|
| Shelter | Bulkheads | 2 x 8 x 1/2 in. Plate | 6 | 5&6 | Aluminum |
| Shelter | Bulkhead & Deadmen | Tie Rod, 7/8 in. Diam. x 6'-6" Long | 4 | 6 | Aluminum |
| Shelter | Bulkhead & Deadmen | Tie Rod, 7/8 in. Diam. x 6'-9" Long | 2 | 8 | Aluminum |
| Shelter | Bulkhead & Deadmen | Tie Rod, 7/8 in. Diam. x 6'-3" Long | 2 | 8 | Aluminum |
| Shelter | Bulkhead & Deadmen | Tie Rod, 5/8 in. Diam. x 11' Long | 8 | 8 | Aluminum |
| Shelter | Bulkhead & Deadmen | Tie Rod, 5/8 in. Diam. x 7'-3" Long | 4 | 8 | Aluminum |
| Shelter | Bulkhead & Deadmen | Tie Rod, 5/8 in. Diam. x 7'-6" Long | 4 | 8 | Aluminum |
| Shelter | Bulkhead & Deadmen | Turnbuckle for 7/8 Diam. Rod | 8 | 8 | Aluminum |
| Shelter | Deadmen | 12 I 17.78 Beam x 10 Ft. Long | 4 | 8 | Aluminum |
| Entranceway | Blast Arrestor Ring | Corrugated Ring, 48 in. Diam. x 2 ft Long | 1 | 1 | Aluminum |
| Entranceway | Hatch Cover | 5 2.32 Beam, Circular, 4'-0" Diam. | 1 | 4 | Aluminum |

Table IX (cont'd)

| Major Assembly | Sub-Assembly | Item Description | No. Req'd | Sheet No. | Material |
|---|---|---|---|---|---|
| Entranceway | Hatch Cover | 5 2.32 Beam, Cross, 47.5 in. Long | 1 | 4 | Aluminum |
| Entranceway | Hatch Cover | 5 2.32 Beam, Cross, 23.5 in. Long | 1 | 4 | Aluminum |
| Entranceway | Hatch Cover | 5 2.32 Beam, Cross, 23.75 in. Long | 1 | 4 | Aluminum |
| Entranceway | Hatch Cover | 5 2.32 Beam, Bearing, 4'-0" Diam. | 1 | 4 | Aluminum |
| Entranceway | Hatch Cover | 34" O.D. x 33" I.D. x 2" Seating Rim | 1 | 4 | Aluminum |
| Entranceway | Hatch Cover | 12 Ga. Sectional Plate, 4'-0" Diam. | 1 | 4 | Aluminum |
| Entranceway | Hatch Cover | Bolt, 1/2" Diam. x 3" Long | 8 | 4 | Hi Str Steel |

Therefore, it is assumed that in spite of the increased costs, a
1-to-1 wall slope will be used. This, of course, also increases
the amount of backfill and compaction. For this excavation, the
increased cost in making the larger 1-to-1 excavation over a 1-to-2
wall is approximately $300. This is about 6 to 7 percent of the
total estimated cost of the materials used in the shelter and is
certainly worthwhile. Detail excavation, backfill, and compaction
calculations are given in Paragraph 5.

Design computations were divided into two main parts,
calculations to determine the corrugated steel gage required and
the end-bulkhead designs.

In determining the proper gage for a given span and rise
pipe arch or underpass, it is necessary to consider seam or joint
failure (failure in shear of bolts or corrugated sheet bearing
failure) and failure as a pipe arch or underpass.

In determining the proper gage size to assure against bolt
shearing or bearing failure, the cross section is considered a ring
of radius equal to one-half the span. It is analyzed then for the
tangential stress in the ring (similar to the stress analysis for a
cylindrical pressure vessel, thin walled). This stress, expressed
as a certain load per foot (pounds per foot) is multiplied by the
safety factor of 4 used in the culvert industry. This final figure,
in pounds per foot, is compared to empirical test data available on
culvert seams, and the proper gage is selected.

In determining failure as a pipe arch or underpass, the
overall loading is expressed as an equivalent height of dead load
or earth cover. This height can then be compared to existing tables
worked out from test data on culvert, and the proper gage can be
selected.

The gages from the two determinations are compared, the
heavier gage ruling the selection.

Results of computations indicate 10-gage sectional plate
is necessary for the shelter pipe arch and 12-gage sectional plate
for the underpass entranceway. Twelve-gage aluminum plate (small
corrugations) is necessary for the vertical hatchway.

For bulkhead design, horizontal pressures were taken at
about three-fourths of the vertical pressure. The work is based on
a plastic design (allowing certain deformation under dynamic load-
ing). Design of deadmen was done following the recommendations of
Reference 2.

It is believed that the design is conservative. Supporting this conclusion is the fact that previous work done on the effects of blast overpressures in buried corrugated metal structures indicates that the designs, using similar procedures, have been extremely conservative. This means that considerably lighter components may be sufficient, greatly affecting assembly time and shelter costs. Considering this, it seems appropriate to recommend that additional work in the form of actually erecting and testing various gage shelters at design loads be undertaken.

4. <u>Construction and Erection Procedure</u>. The construction and erection procedure is divided into five parts presented in order of actual assembly sequence.

a. <u>Excavation</u>. Excavate to a depth of approximately 12 feet the area designated on the excavation plan (Fig. 3). The dimensions given for excavation are a minimum at the 12-foot depth. Slope the excavation walls as necessary to provide for the safety of the men who will be working on the structure. Without experienced judgment available, slope the walls to a 1-to-1 slope for assurance of safety.

In order for the shelter to provide the most effective protection against blast loading, it is essential that the sectional plate pipe arch and also the entranceway structure be supported uniformly throughout their length. The grade must be even and the foundation free from large rocks, roots, organic material, and any other matter which would be detrimental to uniform support of the structure.

If rock is encountered at the 12-foot depth, excavate the foundation area as shown on the excavation plan an additional 8 inches and backfill to the 12-foot depth with granular material.

If at the 13-foot depth unstable material is encountered or the composition is such that unequal settlement of the structure would occur, excavate the foundation area as defined previously to such a depth that when backfilled to the 13-foot depth with suitable material the structure will receive uniform support.

The foundation may be shaped to the contour of the bottom of the structure before assembly, or the structure may be assembled on a flat foundation. In the latter case, after assembly, the backfill material must be tamped well and uniformly under the structure; however, the flat foundation facilitates construction and is, therefore, recommended.

54
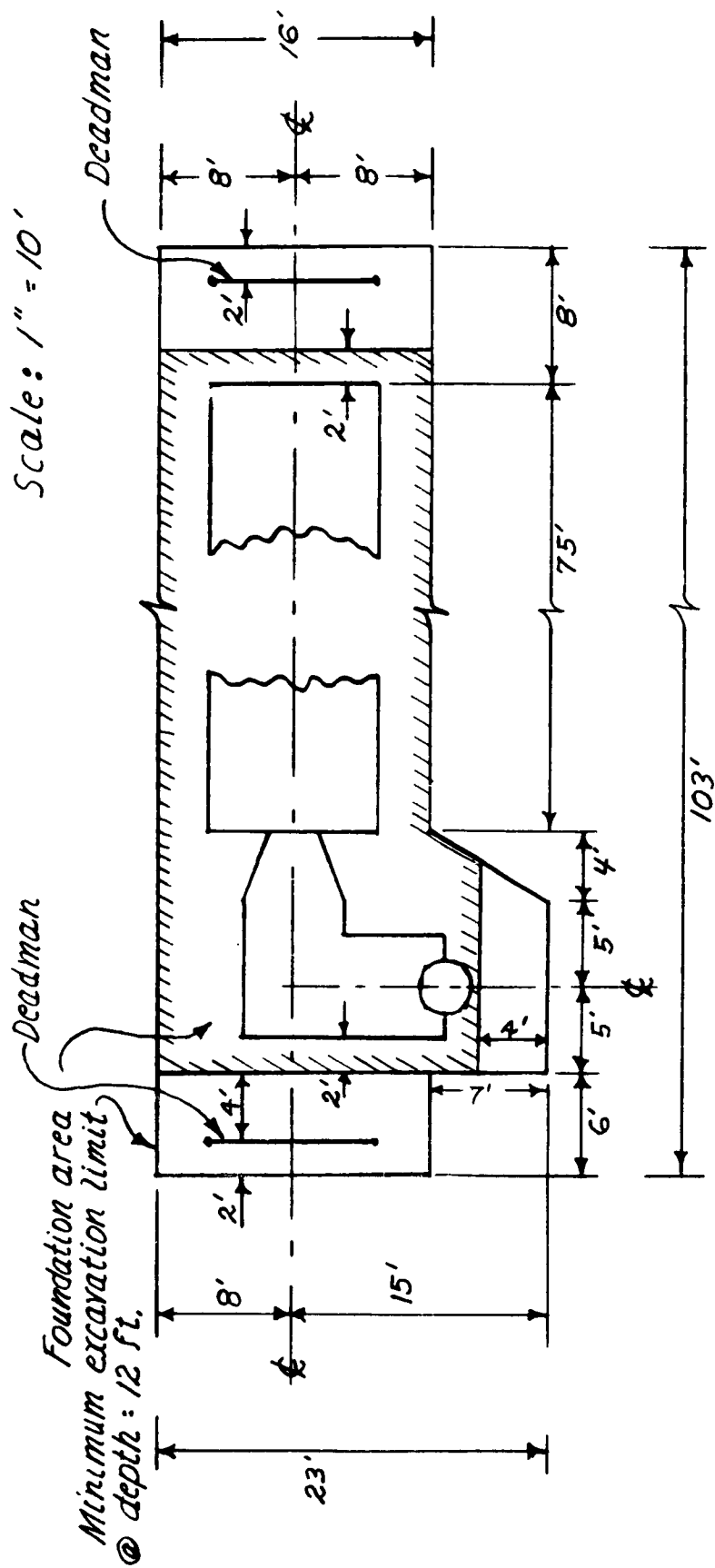


Fig. 3. Excavation plan - metal structures.

b. <u>Assembly of Sectional Plate Pipe Arch</u>. Assemble the sectional plates of the pipe arch in the numerical sequence shown on the shelter development drawing (Exhibit 2, sheet 2). Place plate 1 in position. Place plate 2 in position (inside lap, one corrugation) and insert the longitudinal bolts (do not tighten) by standing on the free edge of plate 1 and lifting the free edge of plate 2. All bolts in the invert plates are to be inserted from the outside. All other bolts are to be inserted from the inside. Align the plates and tighten all the bolts. It is suggested that a chalk line be used to align the invert plates longitudinally. Place plate 3 in position, an inside lap on plate 1 by one corrugation, and insert the transverse bolts (do not tighten). Place plate 4 in position, overlapping plate 2 by one corrugation and overlapping plates 1 and 3 (all inside laps), and insert the longitudinal bolts (do not tighten) using a method similar to that used for plate 2. Insert the transverse bolts, align the plates, and tighten all bolts not previously tightened. Continue this process until all invert plates are in place.

Install the corner plates, 40 through 79. Corner plates lap inside invert plates and lap inside preceding corner plate by one corrugation. Insert enough bolts to hold the corner plates securely, but do not tighten them.

After the corner plates are assembled, starting at the opposite end, place the side and top plates, beginning with 80 and ending with 139. Side plates outside lap corner plates and preceding side plates. Top plates outside lap side plates and preceding top plates. Install only enough bolts to hold the plates firmly in place. Place only enough side plates in position in order to add the next top plate.

When all the plates are in place, install the remaining bolts using drift pins when necessary, and tighten all bolts to a torque of 100 to 200 foot-pounds. When done, go back and check all bolts for tightness.

c. <u>Shelter Bulkhead Assembly (Exhibit 2, Sheets 5 through 7)</u>.

(1) <u>Rear Bulkhead (Sheet 6)</u>. Construct the framework of the bulkhead in a horizontal position, the outside facing up (see the drawings for details). At points C, D, I, and H, do not connect deadman tie rods. Place the appropriate corrugated aluminum sheets on the frame, and install as many 1/4-inch-diameter by 1-1/2-inch-long bolts as are necessary to hold the sheets in place. Tilt the frame to the vertical

position, install the remaining bolts, and bolt the wood braces to the bulkhead according to the details shown in the drawings, using 1/4-inch-diameter by 10-inch-long bolts.

Place the bulkhead into the sectional plate pipe arch so that the face of the bulkhead is 3 inches from the end of the arch. The bulkhead must be positioned so that the corrugated aluminum sheeting is on the outside of the framework.

Place the bottom deadman so that the tie rods from points A and L protrude approximately 1 inch through the deadman. Install the tie rods to points C and J, and bolt to the deadman. Before tightening the bolts, backfill in front of the deadman to the height of the deadman, compacting the material well.

(2) Front Bulkhead (Sheet 5). Assemble the front bulkhead using a procedure similar to that for the rear bulkhead. To allow entranceway clearance for the deadman tie rods, an 8 WF 5.90 is used instead of a 4 $\mathsf{L}$ 2.16 on the right side of the structure. The member EH is omitted, and the wood braces are placed vertically. Also, the top and bottom plates in the center section are installed as shown in the drawings. The tie rods are in two sections. Connect one section to points F, G, M, A, L, and N. After the bulkhead has been placed in the sectional plate pipe arch, add the second section to the anchor rods from points A, L, and N and install the deadman as before. Install the anchor rod to point C.

d. Assembly of Sectional Plate Underpass Shape Entranceway.

(1) Underpass Shape Entranceway Assembly. Assemble the sectional plates of the entranceway in numerical sequence as shown in sections one and two, Exhibit 2, sheet 3. The procedure used for assembling the plates is similar to that used for the shelter proper. The entranceway is attached to the shelter bulkhead in a standard bolted butt connection.

(2) Vertical Entranceway and Entranceway Bulkhead Assembly. Assemble the vertical entranceway and associated bulkhead according to Exhibit 2, sheet 8. Sub-excavate the foundation so that two 10 WF 7.30, two 417.7, and one 3 $\mathsf{L}$ 6 may be installed. Place the lower section of the 36-inch-diameter corrugated metal pipe in place. Install the two 417.7 beams, bolting on the 1-1/2 by 2 angles and the 3 $\mathsf{L}$ 6 brace on the

bottom. Place the two 10 WF 7.30 beams in place, bolting on the 1-1/2 x 2 angles. Connect the two beams at the bottom with a 3 ⊏ 4.1. Attach a sectional plate on each side. Install the remaining 3 ⊏ 4.1's. Insert the 3/4-inch-diameter rods as shown, and tighten them adequately. Backfill to the foundation level. Install the ladder rungs in the 36-inch-diameter corrugated metal pipe. Install the second section of the corrugated metal pipe and the connecting band after the backfill has reached the height of the sectional plate entranceway. Install the second, upper section of ladder rungs in the 36-inch pipe.

    e.    Backfilling. It is preferable that backfill material be granular and drainable if available. Otherwise, use the best suitable local material. The material should be free of organic matter, large stones, foreign matter, etc.

    Place the backfill in 6-inch horizontal layers, and tamp it thoroughly and evenly to obtain uniform compaction. Be especially certain that the material under the sectional plate pipe arch is well compacted. Bring the fill up evenly all around the structure to the elevation of points D and I (see drawings). Adjacent to the bulkheads of the shelter proper, backfill on a slope. Install the remaining deadmen and anchor rods. Compact the earth well in front of the deadman and for a height of 2 feet above it. When the height of the backfill reaches 1-1/2 feet from the top of the sectional plate pipe arch, compaction of the fill may be terminated; however, the remaining backfill should be placed in even layers to the surface of the surrounding ground.

    5.    Estimates. The three following estimates cover cost of the finished material, man-hours involved in the erection of the shelter, and man-hours necessary for the backfill.

    a.    Material Cost. The prefabrication and material cost are based on presently prevalent prices. These prices would probably be reduced to a certain extent in the event of a large-scale shelter program. All prefabrication shop time is rated at $10 per hour. This is a little high but gives a conservative estimate.

    With proper shop equipment, it is estimated that about 16 to 20 man-hours will be necessary to prefabricate, i. e., drilling bolt holes, welding, and cutting beams to required sizes and shapes for shelter bulkheads and entranceway bulkhead. This would cost $160 to $200 for shelter bulkheads and entranceway bulkhead. At present, a typical price per foot of structure, for a sectional plate pipe arch of 10-foot 8-inch span by 6-foot 11-inch rise is $50 per lineal foot, including bolts, unassembled and not

erected F. O. B. destination. This results in $3,750 for a 75-foot structure. The sectional plate underpass entranceway would cost about $30 per lineal foot including bolts, unassembled and not erected F. O. B. destination. This amounts to $420 for that portion of the entranceway. The hatchway would cost approximately $70 assembled into two pieces but not erected. Material and prefabrication costs for hatchway cover total $40. Material costs for the shelter bulkheads and entranceway bulkhead are $200. Therefore, total shelter cost is $4,680. For a 60-man shelter, this is $78 per man.

   b.   Erection Effort. Erection times are given in man-hours. Therefore, total time can be roughly determined by the individual group depending on the number of men available. It is assumed that the labor force will be 60 percent efficient; that is, it will be doing constructive work 36 minutes out of each hour. Erection times are as follows:

   (1) Shelter proper (pipe-arch):  525 man-hours.

   (2) Entranceway (underpass):  90 man-hours.

   (3) Hatchway (circular pipe):  2 man-hours.

   (4) Total front and rear shelter bulkhead:
        17.1 man-hours.

   (5) Entranceway bulkhead:  11.4 man-hours.

   (6) Hatchway cover:  0.8 man-hours.

   (7) Deadman attachments (total front plus rear):
        4.8 man-hours.

   (8) Backfill time:  828 man-hours.

   (9) Compacting time:  268 man-hours.

   (10) Total time from start to finish (excluding
        excavation time):  1747.1 man-hours.

   Details of the erection time required for various parts of the shelter are given in the following analysis.

Shelter Erection Times

I. Front Bulkhead - Total time 8.3 man-hours

A. Placing bolts at 1 hr/bolt

1. Joints A, F, G, and L have 6 bolts per joint totaling 24 bolts.

2. Joints C, D, I, and J have 3 bolts per joint totaling 12 bolts.

3. Joints B, E, H, and K have 7 bolts per joint totaling 28 bolts.

Therefore, total bolts = 64 bolts or 6.4 man-hours total.

B. Application of Sectional Plates

At 1 hr/100 sq ft.

Area = 58 - (6.5)(3) = 38.5 sq ft.

therefore, time = 0.385; Assume 0.5 man-hours

C. Application of Wood Braces

Total of 4 bolts at 0.1 hr/bolt results in 0.4 man-hours

D. Raising of Assembled Bulkhead into Position

1 man-hour

E. Total Time

6.4 + 0.5 + 0.4 + 1.0 = 8.3 man-hours

II. Rear Bulkhead - Total time 8.8 man-hours

In addition to time of front bulkhead, one additional sectional plate and one beam must be added.

A. Sectional Plate Addition 0.3 man-hours

B. Additional Beam: Two additional bolts required at 0.1 hr/bolt giving 0.2 man-hours.

C. Total Time

$8.3 + 0.3 + 0.2 = \underline{8.8 \text{ man-hours}}$

III. <u>Entranceway Bulkhead - Total 11.4 man-hours</u>

    A. A total of 84 bolted connections are made at a rate of 0.1 hour per bolt, giving 8.4 man-hours.

    B. In addition, 3 man-hours are allowed for additional excavation and backfill required.

    C. Total is, therefore, 11.4 man-hours.

IV. <u>Hatchway Cover - 0.8 man-hours</u>

8 bolts at 0.1 hour per bolt gives 0.8 man-hours

Plus 0.5 man-hours for hinge attachment, therefore, total time is 1.3 man-hours

V. <u>Deadman Attachment - 4.8 man-hours</u>

    A. Front

        32 bolted connections at 0.1 hr/bolt = 3.2 man-hours

    B. Rear

        16 bolted connections at 0.1 hr/bolt = 1.6 man-hours

    C. Total Time = 3.2 + 1.6 = 4.8 man-hours

VI. <u>Shelter Proper (Pipe Arch)</u>

Consider rate of 7 man-hours/lineal foot of length, including bolting, unloading, setting plates in place, etc.

75 ft of shelter gives a total of 525 man-hours

VII. <u>Entranceway (Underpass)</u>

Consider 5 man-hours per lineal foot of length and 18 feet long.

Total Time = 90 man-hours

VIII.  Vertical Hatchway Pipe Connection

2 man-hours

IX.  Backfill and Compaction Time

Assumptions:

A.  Compaction Rate = .75 cu yd/man-hour

B.  Backfilling Rate (for medium soil) = 1-1/2 cu yd/man-hour

C.  Wall Slope = 1:1 or 45°

Backfill Vol. = Excavation Vol. - Shelter Vol.

$$V_B = V_E - V_S$$

(See Fig. 3)

$$V_E = (103)(16)(12) + (7)(10)(12) + 7(2)(12) + \frac{1}{2}(12)(12)(103)$$

$$+ (\frac{1}{2})(12)(12)(16)(2) + (\frac{1}{2})(12)(12)(110)$$

$$\begin{array}{cc} \text{(Vert. Wall Vol.)} & \text{(Sloped Position)} \\ V_E = [19776 + 840 + 168] & + [7416 + 2304 + 7920] \end{array}$$

$$V_E = 20,784 + 17,640$$

$$V_E = 38,424 \text{ cu ft, or } 1,423.1 \text{ cu yd}$$

$$V_S = V_{Ent.} + V_{Shelt.} + V_{Hatchway}$$

$$V_S = V_{Ent.} + V_{S_S} + V_H$$

$$V_S = (14)(35) + (75)(58) + (\frac{1}{2})(6.5)(7.08)$$

$$+ (4.5)(7.08) = 4,895 \text{ cu ft}$$

$$\text{or } 181.3 \text{ cu yd}$$

Therefore, $V_B = 1423.1 - 181.3 = \underline{1241.8 \text{ cu yds Backfill vol.}}$

Compaction Volume = $V_C$

$$V_C = (16)(7.0)(75) + (7)(7)(\tfrac{1}{2})(75)(2) + (16)(2)(6.5)(8)$$
$$+ 10(5.5)(6.5) + (16)(6.5)(12) - V_S$$

$$V_C = 8400 + 3675 + 208 + 2496 + 357.5 - 181.3$$

$$V_C = \underline{6203.2 \text{ cu ft or } 229 \text{ cu yd}}$$

Consider 200 cu yd at 5.25 ft height (3/4 of height to top of sectional plate)

Therefore:

$$\frac{1241.8}{1.5} = \frac{\text{Backfill Vol.}}{\text{Backfill Rate}} = \frac{\text{yd}^3}{\text{yd}^3/\text{man-hr}}$$

<u>828 man-hours backfilling</u>

$$\frac{200}{.75} = \underline{268 \text{ man-hours compaction}}$$

1096 man-hours backfilling plus compaction time
Round off to 1100 man-hours

X.  Total Shelter Construction Time

```
          2.0
          8.3
          8.8
         11.4
          1.3
          4.8
        525.0
         90.0
      1,096.0
      1,747.6 Man-hours
```

XI.  <u>Sample Calculation for Determination of Construction Time in Days for a Specific Crew</u>

Consider 6-man crew working 8-hour days.

Then:  6 x 8 = 48 man-hours per day.

or $\frac{1747.6}{48} = \underline{36.40 \text{ working days}}$

## REFERENCES

1. Department of the Army, "The Effects of Nuclear Weapons,"
   Pamphlet No. 39-3 (Washington, D. C.: U. S. Government
   Printing Office, April 1962).

2. Department of the Army, Engineer Troop Protective Construction
   (Nuclear Warfare), TM5-311 (Washington, D. C.: U. S. Gov-
   ernment Printing Office, August 1961).

3. Peurifoy, Robert L., Estimating Construction Costs (New York,
   N. Y.: McGraw-Hill Book Co., Inc., 1953).

4. Cooper, H., Building Construction Estimating (New York, N. Y.:
   McGraw-Hill Book Co., Inc., 1959).

5. ARMCO International Corp., Handbook of Drainage and Construction
   Costs (Middletown, Ohio: 1955).

6. Dallavia, L., Estimating Production and Construction Costs
   (Houston, Texas: Dallavia Co., 1954).

7. Aluminum Association, "Aluminum Construction Manual" (New York,
   N. Y.: 1959).

D

3' DIAM — ——— SECTION 3

9'

——— SECTION 2

SECTION 1

——— SHELTER

C

EARTH SURFACE —————

——— 4' DIAM × 2' CORR RING

——— CRUSHED ROCK

5'

5"

B

75'

9'

84'

A

| | | |
|---|---|---|
| 3 | 2 | 1 |

SECTION B

3'

SECTION 2

SECTION 1

4 DIAM × 2' CORR RING

CRUSHED ROCK

RISE 6'-11"
RISE 6'-6"

11'-2" 12'-2"

2'-2"

SPAN 5-9'

SPAN 10'-6'



| APPLICATION | | UNLESS OTHERWISE SPECIFIED | | U. S. ARMY |
|---|---|---|---|---|
| NEXT ASSY | USED ON | DIMENSIONS ARE IN INCHES TOLERANCES ON FRACTIONS DECIMALS ANGLES | | ENGINEER RESEARCH AND DEVELOPMENT LABORATORIES CORPS OF ENGINEERS FORT BELVOIR, VA |

BREAK SHARP EDGES TO
ALL FILLETS TO

REVIEWED FOR PRODUCTION:

**CIVILIAN GROUP SHELTER
METAL**

MATERIAL
CORRUGATED STEEL
(SECTIONAL PLATE EXCEPT
FOR VERTICAL PIPE)

| DRAWN BY | TRACED BY | CHECKED BY |
|---|---|---|

| CAGE (IDENT. NO.) | SIZE | | |
|---|---|---|---|
| 97403 | D | 13208E 5648 | |

SCALE: 1/4"=1'-0"   SHEET 1 OF 6

EARTH COVER SURFACE

5'

ENTRANCE END

## ELEVATION

76'

| 79 | 78 | 72 | 71 | 70 | 69 | 68 | 67 | 60 | 59 | 58 | 57 | 56 | 55 | 48 | 47 | 46 | 45 | 44 |
| 81 | 84 | 87 | 90 | 93 | 96 | 99 | 102 | 105 | 108 | 111 | 114 | 117 | 120 | 123 | 126 | 129 | 132 | 135 |
| 82 | 85 | 88 | 91 | 94 | 97 | 100 | 103 | 106 | 109 | 112 | 115 | 118 | 121 | 124 | 127 | 130 | 133 | 136 |
| 80 | 83 | 86 | 89 | 92 | 95 | 98 | 101 | 104 | 107 | 110 | 113 | 116 | 119 | 122 | 125 | 128 | 131 | 134 |
| 77 | 76 | 75 | 74 | 73 | 66 | 65 | 64 | 63 | 62 | 61 | 54 | 53 | 52 | 51 | 50 | 49 | 42 | 41 |
| 37 | 35 | 33 | 31 | 29 | 27 | 25 | 23 | 21 | 19 | 17 | 15 | 13 | 11 | 9 | 7 | 5 | 3 | |
| 39 | 38 | 36 | 34 | 32 | 30 | 28 | 26 | 24 | 22 | 20 | 18 | 16 | 14 | 12 | 10 | 8 | 6 | 4 |

TOP ¢

BOTTOM ¢

4'

## DEVELOPED PLAN

**ELEVATION**

5'

ENTRANCE END

**END VIEW**

TOP PLATE

SIDE PLATE — SIDE PLATE

CORNER PLATE — — CORNER PLATE

INVERT (BOTTOM) PLATES

FIRST RING
CROSS SECTION, SHOWING
PLATE LAPPING ORDER

**DEVELOPED PLAN**

| 58 | 57 | 56 | 55 | 48 | 47 | 46 | 45 | 44 | 43 | 9 R INCHES, | CORNER PLATE |
| 108 | 111 | 114 | 117 | 120 | 123 | 126 | 129 | 132 | 135 | 18 R | SIDE PLATE |
| | | | | | | | | | | 21 R | TOP PLATE |
| 112 | 115 | 118 | 121 | 124 | 127 | 130 | 133 | 136 | 139 | | |
| 107 | 110 | 113 | 116 | 119 | 122 | 125 | 128 | 131 | 134 | 18 R | SIDE PLATE |
| 61 | 54 | 53 | 52 | 51 | 50 | 49 | 42 | 41 | 40 | 9 R | CORNER PLATE |
| 19 | 17 | 15 | 13 | 11 | 9 | 7 | 5 | 3 | 1 | 18 R | |
| | | | | | | | | | | | INVERT PLATES |
| 20 | 18 | 16 | 14 | 12 | 10 | 8 | 6 | 4 | 2 | 15 R | 4 32' |

4' 2' ENTRANCE END

:TOP PLATE

SIDE PLATES

CORNER PLATES

BOTTOM (INVERT) PLATE

END VIEW OF SECTION 1 SHOWING
PLATE LAPPING (OUTER OR INNER
LAP) ORDER OF END RING



15 Ft SIDE PLATE

9 Ft CORNER PLATE

9 Ft BOTTOM PLATE

9 Ft CORNER PLATE

15 Ft SIDE PLATE

21 Ft TOP PLATE

SECTION 1

-TOP PLATE

- SIDE PLATES

- CORNER PLATES

- BOTTOM (INVERT) PLATE

END VIEW OF SECTION 1 SHOWING
PLATE LAPPING (OUTER OR INNER
LAP) ORDER OF END RING

9'

TOP PLATE

SIDE PLATES

CORNER PLATES

BOTTOM (INVERT) PLATE
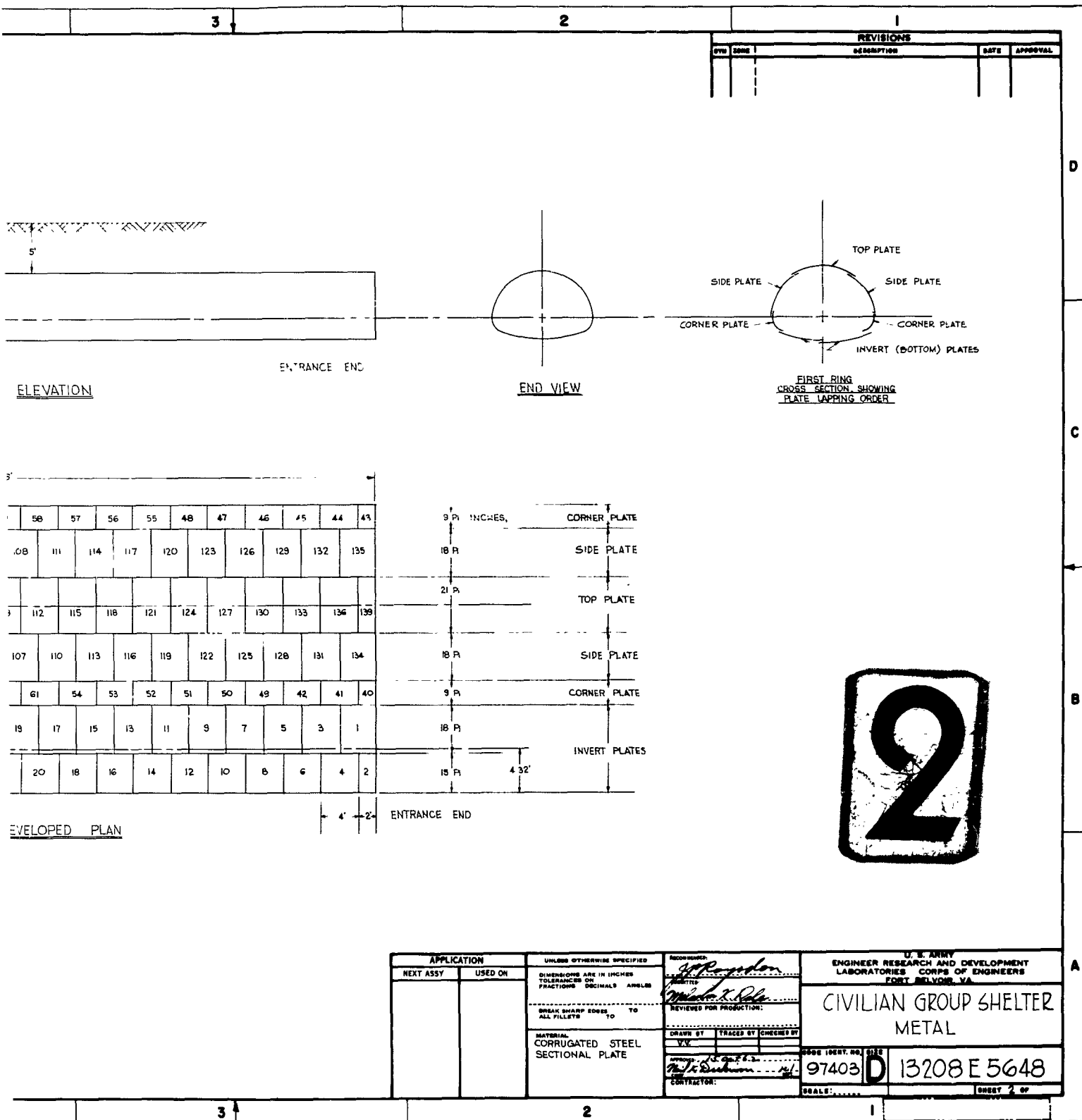
END VIEW OF SECTION 2 SHOWING
PLATE LAPPING (OUTER OR INNER
LAP) ORDER OF END RING

4

| 2 | 15 | 17 | 15 Ft SIDE PLATE |
| 9 | 8 | 5 | 4 | 9 Ft CORNER PLATE |
| 6 | 2 | 1 | 9 Ft BOTTOM PLATE |
| 10 | 7 | 3 | 9 Ft CORNER PLATE |
| 11 | 14 | | 15 Ft SIDE PLATE |
| 13 | 16 | 18 | 21 Ft TOP PLATE |

2'   4'

SECTION 2

**2**

3   2   1

36" DIA

29 5" R

11'-2"

7'-9"

6'-6"

12" WIDE
CONNECTING
BAND LOCATION

7'-3"

2

1

DEVELOPMENT

VERTICAL HATCHWAY, SECTION 3
ENTRANCE
CORRUGATED STEEL PIPE

A

CORR
PLATE

A

JOINT #1

A

TOP €

A

JOINT #2

JOINT #1

JOINT #2

BEARING BEAM
5 ⊏ 2.32

CORRUGATED
STEEL PLATE

SEATING RIM
(HOLDS COVER IN PIPE)

CROSS FRAME
5 ⊏ 2.32

CIRCULAR FRAME
5 ⊏ 2.32

COVER PLATE HOLDING BOLTS
(SEE DETAILS)

HATCHWAY   COVER

48"

JOINT #1

SECTION A-A

CORR COVER PLATE

JOINT #2

TOP & BOTTOM

SECTION A-A

CIRCULAR BEAM A-A
5 C 2.32 ALUMINUM 2014-T6
1 REQ'D.

CROSS BEAM
5 C 2.32 ALUMINUM 2014-T6
1 REQ'D.

½ DIA.

CROSS BEAMS
5 C 2.32 ALUMINUM 2014-T6

I REQ'D.

REQ'D.

4-0'

BEARING BEAM SECTION A-A
5 C 2.32 ALUMINUM 2014-T6

53" I.D.

SEATING RING SECTION A-A
ALUMINUM 2014-T6

94" O.D.

— BEARING BEAM
5 C 2.32

— SEATING RIM
(HOLDS COVER IN PIPE)

— CROSS FRAME
5 C 2.32

— CIRCULAR FRAME
5 C 2.32

48"

CORRUGATED ALUMINUM SHEET
ALUMINUM ALLOY 2014-T6
2" DEPTH 6" PITCH

4"x 4" x 6'-8" WOOD BRACE
SEE DETAIL "D"

SEE DETAIL "C"

D

SEE DETAIL "B"

SEE DETAIL "A"

SEE DETAIL "A"

C

SEE DETAIL "C"

SEE DETAIL "B"

SEE DETAIL "C"

B

FRONT – SHELTER BULKHEAD
SCALE: 2" = 1'-0"

A

2 4"x 4"x 6'-8" WOOD BRACE
SEE DETAIL -"D"

SEE DETAIL -"C"

SEE DETAIL-"B"

SEE DETAIL -"A"

SEE DETAIL -"A"

SEE DETAIL-"B"

SEE DETAIL-"C"

1'-0"   1'-0"   6"   6"   1'-0"   1'-0"

F
F

C          C

2

NOTE.
THE RADIAL DIMENSIONS OF THE
CORRUGATED ALUMINUM SHEETS ARE 1/4"
LESS THAN THE INSIDE RADIAL DIMENSIONS
OF THE SECTIONAL PLATE PIPE ARCH

ONT - SHELTER BULKHEAD
SCALE: 2" = 1'-0"

CIVILIAN GROUP SHELTER
METAL

BREAK SHARP EDGES TO
ALL FILLETS TO

MATERIAL

DRAWN BY   TRACED BY   CHECKED BY

CODE IDENT. NO.  SIZE
97403   D   13208E5648

SCALE:          SHEET 5 OF

D

C

B

A

5      4      3

40"(CENTER SHEET)

℄ OF PIPE ARCH

2"

⅝

1½

2"

D

3½"

D   D

1'-0"   1'-0"

1'-0"

1'-0"

1'-0"

1'-0"

1'-0"

1'-0"

G"

G"

1'-0"

1'-0"

G"

G"

1'-0"

C

SEE DETAIL-"B"

SEE DETAIL

1½"

J

H   H

J

A   A

SEE DETAIL "A"

SEE DETAIL-"A"

4"x4"x G'-8" WOOD BRACE
SEE DETAIL-"D"

SEE DETAIL-"A"

SEE DETAIL-"C"

1'-0"

B

1⅛"

1'-0"

SEE DETAIL

SEE DETAIL-"B"

G

C   C

G

SEE DETAIL-"C"

E   E

⅝

1"
4

2"

2"

4'-1¼" (LEFT SHEET)

REAR - SHELTER BULKHEAD
SCALE: 2"=1'-0"

A

5      4      3

— 40" (CENTER SHEET) —

₵ OF PIPE ARCH

2"

1½"

— 1'-0" — 1'-0" —

1'-0"

1'-0"

1'-0"

— 1'-0" — 1'-0" — 6" — 6" — 1'-0" — 1'-0" —

1½"

SEE DETAIL -"B"

1'-0"

SEE DETAIL -"A"

4" x 4" x 6'-8" WOOD BRACE
SEE DETAIL -"D"

SEE DETAIL -"C"

NOTE
THE RADIAL DIMENSIONS OF THE
CORRUGATED ALUMINUM SHEETS ARE ¼"
LESS THAN THE INSIDE RADIAL DIMENSIONS
OF THE SECTIONAL PLATE PIPE ARCH

SEE DETAIL -"B"

C    C

SEE DETAIL -"C"

2"

— 3'-10¼" (RIGHT SHEET) —    ¼"

R - SHELTER BULKHEAD
SCALE. 2" = 1'-0"

D    C    B    A

SECTION "A-A"
SCALE 3"=1'-0"

SECTION "B-B"
SCALE 3"=1'-0"

SECTION "C-C"
SCALE 3"=1'-0"

SECTION "D-
SCALE 3"=1'-0"

SECTION "F-F"
SCALE 3"=1'-0"

SECTION "G-G"
SCALE 3"=1'-0"

SECTION "H-H"
SCALE 3"=1'-0"

SECTION "J-J"
SCALE 3"=1'-0"

DETAIL "A"
SCALE 2"=1'-0"

DETAIL "B"
SCALE 2"=1'-0"

DETAIL "C"
SCALE 2"=1'-0"

**SECTION "C-C"**
SCALE 3"=1'-0"

L 3"x 3"x 2"x ⅜"
4I 2 64
L 3"x 3"x ⅜"
8WF 5 90

**SECTION "D-D"**
SCALE 3"=1'-0"

4E 2.16
8WF 5 90

**SECTION "E-E"**
SCALE 3"=1'-0"

8WF 5 90

**SECTION "H-H"**
SCALE 3"=1'-0"

2-2¼"x⅛"x½"
L 3½"x 3"x ⅜"
4I 2 64

**SECTION "J-J"**
SCALE 3"=1'-0"

2 ½"x 8"x 2"

**DETAIL-"B"**
SCALE 2"=1'-0"

**DETAIL-"C"**
SCALE 2"=1'-0"

4E 2 16

**DETAIL-"D"**
SCALE. NONE

WASHER
DOUBLE NUT
WOOD BRACE
4 x 4
¾" DIA x 10" LG BOLT
3"

ALUMINUM 12 I 17.28 10' LONG

DETAIL "A"
SCALE: 6" = 1'-0"

10 WF 7.30 ALUMINUM

| APPLICATION | | UNLESS OTHERWISE SPECIFIED |
|---|---|---|
| NEXT ASSY | USED ON | DIMENSIONS ARE IN INCHES TOLERANCES ON FRACTIONS DECIMALS ANGLES |
| | | BREAK SHARP EDGES TO ALL FILLETS TO |
| | | MATERIAL |

D

STEEL 7' LONG — 4"

L1½×2×¼ 6'-6"L

12 GA. COPR #L
ALUMINUM

¾ ROD,
STEEL

DETAIL 'A'

23¾"

27¾"

—ENDS OF SHELTER

ALUMINUM 12 I 17.28 10' LONG

⑤ ①
⑥ ②
⑦ ③
⑧ ④

ENTRANCEWAY

18'          6'

DRILL HOLES IN I BEAM AND BOLT RODS USING A FLAT WASHER.
BUILD UP ROD ENDS OR INCREASE ROD DIAM BY ⅛".

C

10 WF 7.30 ALUMINUM

½" @ 12"
3C4 1

1½×2×⅜ 6'-6"

3'-9"

2'-0"

2'-0"

2'-0"

CORR. SHEET
G G-6" WIDE
MINUM

3'-3"

3'-0"

| ROD NO | DIAM | LENGTH | NOTES | |
|---|---|---|---|---|
| 1 | 7/8 | 6'-6" | MATERIAL | |
| 2 | 5/8 | 6'-9" | REAR | ALL RODS ALUMINUM |
| 3 | 5/8 | 6'-3" | | |
| 4 | 7/8 | 6'-6" | | |
| 5 | 5/8 | 18'-6" | | |
| 6 | 5/8 | 18'-3" | FRONT/USE 7'-3" & 7'-6" RODS WITH | |
| 7 | 5/8 | 18'-3" | AN 8'-0" ROD WITH TURN- | |
| 8 | 5/8 | 18'-6" | BUCKLE CONNECTORS) | |

TIE ROD SCHEDULE

2

B

A

METAL AND TIMBER STRUCTURES

by

John D. Morelli

1.    Design Considerations.

        a.    Design Pressures.  The structure is assumed to be
under 5 feet of earth cover and is designed to resist a peak dynamic
overpressure of 20 psi.  The design is based on the ultimate strength
of the materials involved and the assumption of an equivalent static
fluid pressure of 23.5 psi.

        b.    Design Stresses.  Since the structure is allowed to
deform plastically, the dynamic load factor and the shape factor of
steel beams tend to cancel so that steel beams may be designed with
an equivalent static stress in bending equal to the dynamic yield
stress.

        Wood is a relatively brittle material so that the
shape factor for wood beams may be assumed to be one.  Therefore, a
dynamic load factor of 1.5 is applied to wood beams loaded directly
by the dynamic load.

        It is assumed that the high soil pressure allowed in
this design is not unreasonable for dynamic loading.

        c.    Size, Shape, and Material of Structure.  The size of
the shelter is based on a requirement of 10 square feet of floor
area per person and a capacity of 60 people.  The designed struc-
ture has 67 cubic feet per person.  The adequacy of this volume is
dubious;  however, should it be proved insufficient, the unit volume
per person may be increased by either decreasing the rated capacity
of the structure or by lengthening the structure.

        The arch-shape shelter using structural tees and tim-
ber lagging was selected because of its simplicity and ease of con-
struction.  This shelter is similar to the one designed by Newmark,
etal (Reference 1).

        At the bulkheads, a horizontal arch is used to trans-
mit the lateral loads to the timber sills.  The use of a wide-flange

beam was investigated but the beam proved to be too heavy. Also the use of a horizontal truss was investigated. This structure is light enough but has too many joints. The shape of the chosen arch is such that bending stresses are a minimum.

The combination timber-steel sills are designed as continuous footings to reduce differential settlement of the arches.

All timber used in the shelter should be treated with a wood preservative.

d. Behavior of Structure Under Load. Since the horizontal pressure is less than the vertical pressure on the structure, there is likely to be transverse lateral deformation of the ST 4 WF 12 arches; however, a small deformation will increase the passive soil pressure and again bring the arch into equilibrium. Deformation of the horizontal arches is likewise resisted by passive soil pressure.

It is expected that there will be some crushing of the timber sills, but this is not detrimental to the function of the shelter.

e. Alternatives. Because there was insufficient time available to design an entranceway for this shelter, a previously designed entranceway is used; however, it appears feasible to design an entranceway using a type of construction similar to that used for the main shelter. This type of construction would reduce the number of man-hours required for the erection of the entranceway.

The bulkhead arch lies outside the shelter, but it is possible to translate the arch so that it lies within the shelter, or the arch may be inverted which would have the same effect. The result would be to decrease the amount of excavation necessary to install the structure.

The shelter is constructed of steel and timber. Because of the lightness of aluminum, it may be well to consider it in place of the steel. It is also possible to replace the timber lagging with a cellular-shaped aluminum extrusion.

2. Construction Procedure.

a. Excavation. Excavate to a depth of 14 feet below existing ground and to a length and width that will allow 3 feet clear all around the structure. Where the foundation material is such that the sills would not receive uniform support, sub-excavate

as necessary and backfill. The cut slopes will be governed by the type of soil encountered, but they should be as steep as possible to reduce the amount of excavation.

b. <u>Assembly of Shelter</u>. Assemble the combination timber and steel sills as shown on the drawings, placing them parallel and at a distance of 16 feet center-to-center. Backfill along the edges of the sills to the height of the sills to prevent movement during construction.

Erect the ST 4 WF 12 arches, allowing the two halves of each arch to butt at the top against the ridge beam. Insert the lag screws in the base plates and install four 1/2- by 4-1/2-inch bolts at the top of each arch. Splice the 2-by-10's with two 1-by-6's and 6d nails.

Bolt the 10 WF 25's to the arches at the ends of the shelter. Construct the 4 WF 13 horizontal arches, bolting together the two halves of each arch with two 1/2- by 2-inch bolts. Install the 1-1/2-inch-diameter rods and then the 1-3/4-inch-diameter tie rods.

Starting at the bottom of each side of the arches, install all the 4- by 8-inch by 3-foot 11-1/2-inch timbers. The earth backfill will hold these members in place.

c. <u>Assembly of Entranceway</u>. The entranceway used for this shelter is the same as the entranceway used for the sectional plate, pipe arch shape shelter described in Appendix C.

d. <u>Backfill</u>. The excavated material may be used for backfill. Backfill the floor inside the shelter to the level of the top of the sills. Outside the shelter, place the soil in 6-inch layers and compact it. Compact the backfill to the elevation of the top of the shelter. Above this elevation, the soil should be placed in even layers but need not be compacted.

3. <u>Time Analysis</u>.

a. <u>Backfill</u>. In order that the cost and construction time of the various shelters described in this report might be compared, a standard side slope and backfill rate are used. In practice, each particular situation will be different and the number of man-hours required for compaction and backfill will have a wide range of values. This is demonstrated by the graph included in the calculations.

The basic assumption that there would not be equipment available for the backfilling operation and that it would all have to be done by hand considerably increases the length of the task. For hand backfilling, the number of man-hours required for constructing the shelter is negligible compared to the number of man-hours required for backfilling and compaction.

Excavation

Assume 1:1 side slopes

Average depth = 13 feet

Area at bottom        = 22 x 60 = 1320 ft$^2$
Area at top           = 48 x 86 = 4128 ft$^2$
Area 8 feet from bottom = 38 x 76 = 2888 ft$^2$

$$V = \frac{1320 + 4128}{2} \times \frac{13}{27} = 1310 \text{ C.Y.}$$

Backfill

Volume of shelter      = 32$\pi$ x 40 = 4020 ft$^3$
Volume of entranceway = 18 x 35   =   630 ft$^3$
                                           4650

                     ÷ 27   =   172 C.Y.

Total backfill = 1310 - 172 = 1138 C.Y.

Compacted backfill

$$V = \frac{1320 + 2888}{2} \times \frac{8}{27} - 172 = 622 - 172 = 450 \text{ C.Y.}$$

Rate for backfill     = 1.5 C.Y./M.H.
Rate for compaction = 0.75 C.Y./M.H.

Man-hours for backfill     =    759
Man-hours for compaction =    600
                  Total     1359
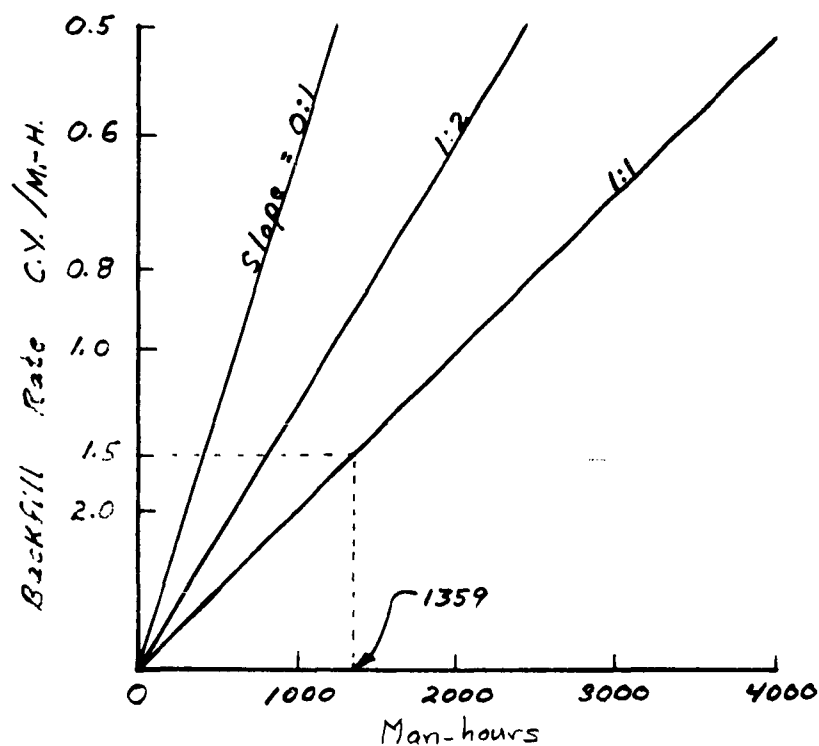
Fig. 4. Man-hours for backfill and compaction - metal-timber structures.

b. **Erection of Shelter and Entranceway.**

Shelter

| | | |
|---|---|---|
| Assemble sills | | 11-1/4 man-hours |
| Erect arches and ridge beam | | 21-3/4 |
| Install bulkhead arch and vertical beams | | 4-1/2 |
| Install 4x8 timbers | | 15 |
| | Sub-total | 52-1/2 man-hours |

| | | |
|---|---|---|
| Entranceway | | 103 man-hours |
| | Sub-total | 156 man-hours |

| | | |
|---|---|---|
| Backfill and compact | | 1359 man-hours |
| | Total | 1515 man-hours |

4.  Cost.

Shelter

| | | | |
|---|---|---|---|
| Timber | 5641 B.F. | at $0.26 | $1469.26 |
| Steel (large items) | 6684 lb | at 0.07 | 467.88 |
| Steel (small items) | 1530 lb | at 0.08 | 122.40 |
| Steel Fabrication | | | 100.00 |
| | | Sub-total | $2159.54 |
| Entranceway | | | $ 730.00 |
| | | Total | $2889.54 |

Table X.  Bill of Materials, Steel and Timber Structures

(a) Timber

| Item | Length (ft) | B.F./Item | Number | B.F. |
|---|---|---|---|---|
| 6" x 6" | 8 | 24 | 4 | 96 |
| " | 11.5 | 34.5 | 8 | 276 |
| " | 12 | 36 | 4 | 144 |
| 2" x 10" | 14.5 | 24 | 2 | 48 |
| " | 12 | 20 | 1 | 20 |
| 4" x 8" | 4 | 10-2/3 | 475 | 5067 |
| | | | Total: | 5651 |

(b) Steel

| Item | Length (ft) | lb/Item | Number | Weight (lb) |
|---|---|---|---|---|
| 7 ⊏9.8 | 3.5 | 34.3 | 8 | 274 |
| " | 12 | 117.5 | 12 | 1412 |
| ST 4 WF 12 half-arch | 12.5 | 150.5 | 18 | 2710 |
| 4"x4"x1/2" L half-arch | 12.5 | 160.5 | 4 | 642 |
| 4 WF 13 fab. half-arch | 9.2 | 125 | 4 | 500 |
| 10 WF 25 | 7.35 | 183.5 | 4 | 734 |
| " | 8.25 | 206 | 2 | 412 |
| | | | Total: | 6684 |
| 6-1/2"x8"x1" PL | - | 14.7 | 22 | 324 |
| 8"x8"x1" PL | - | 18.1 | 22 | 398 |
| 1"x12-1/2" bolt | - | 3.48 | 86 | 300 |
| 1/2"x4" lag screw | - | 0.31 | 88 | 27 |
| 1/2"x4-1/2" bolts | - | 0.344 | 44 | 15 |
| 1/2"x3" bolts | - | 0.26 | 4 | 2 |
| 1-1/2" diam. bar | 3.25 | 19.5 | 4 | 78 |
| " | 4.5 | 27 | 2 | 54 |
| 1-3/4" diam. bar | 17.25 | 141 | 2 | 282 |
| Miscellaneous steel | - | - | - | 50 |
| | | | Total: | 1530 |

REFERENCES

1. Newmark, N. M., J. W. Briscoe, and J. L. Merritt, Analysis and Design of Flexible Underground Structures (Volume I) (Urbana, Illinois: Contract DA-22-079 Eng-225, 1 May 1960).

2. Marshall, R. G., and J. E. Pleasants, "Hasty Personnel Shelter for Protection from Atomic Effects," Report 1571-TR (Ft. Belvoir, Va.: U. S. Army Engineer Research and Development Laboratories, 9 March 1959).

3. Norris, C. H., R. J. Hansen, M. J. Holley, Jr., J. M. Biggs, S. Namyet, and J. K. Minami, "Structural Design for Dynamic Loads," McGraw-Hill Book Co., 1959.

3'-6" | 12'-0" | 12'-0" | 12'-0" | 3'-6"

7☐98

11'-6" | 8'-0" | 12'-0" | 11'-6"

6"x6" LA

11'-6" | 12'-0" | 8'-0" | 11'-6"

5'-9" SPAN , 6'-6" RISE , 12 GAGE
SECTIONAL PLATE UNDERPASS
SHAPE - CONSULT DWG
FOR DETAILS

7☐98 BOTH SIDES - SEE ABOVE PLANS FOR LENGTHS OF MEMBERS
2-6"x6" TIMBER SILL
1"∅ TIE BOLTS, 1'-0" OC

1½"∅ TIE

DETAIL "A"

DETAIL "B"

DETAIL "D"

∠4x3x½

6½x8x1 PL

SPLICE WITH 2-1"x6"x3'-0"
2"x10" RIDGE BEAM
ST4 WF12 @ 4'-0" OC
8x8x1 PL

4 WF 13

10 WF 25

1½"∅

1½"∅

∠4x3x½

DETAIL "C"

12'-0"

10 WF 25

1½"∅

4"x8"
PLANKING

3" DIA CMP
12 GAGE

ENTRANCE

1'-6" | 4'-0"

8 @ 4'-0" = 32'-0"

4'-0" | 1'-6" | 3'-0"

12'-0"

43'-0"

4'-1½"
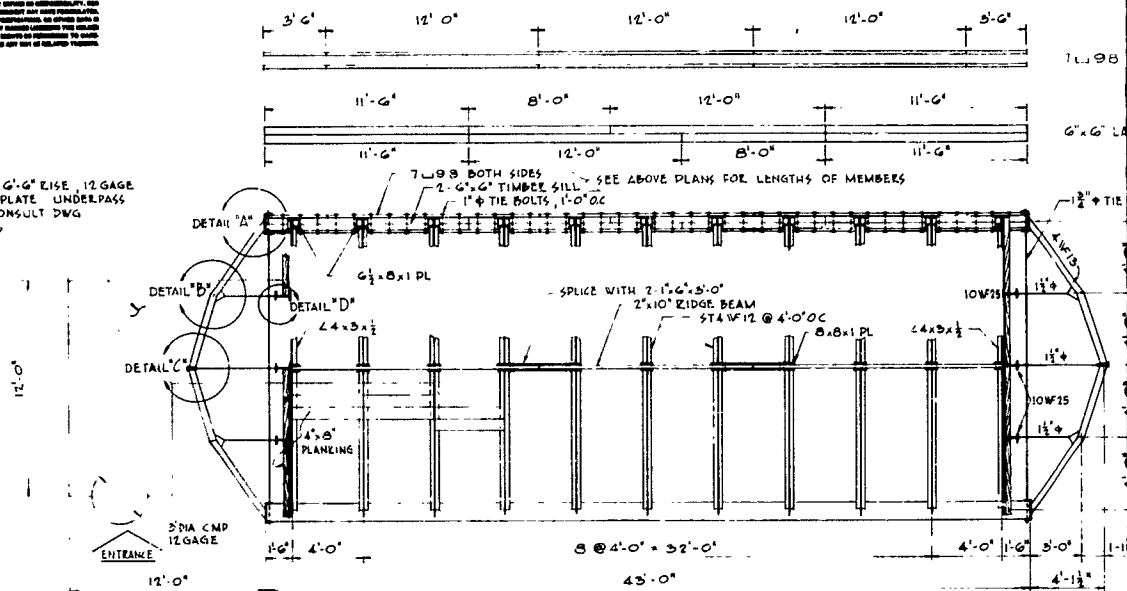
## FOUNDATION , FLOOR & FRAMING PLAN
### SCALE ½" = 1'-0"

2"x10" RIDGE BEAM
8"x8" x 1" PLATE
½"∅ x 4½" BOLTS

4"x8"x3'-11" PLANKING

10 WF 25

10 WF 25

10 WF 25

8'-0" ℄

4"x8"x3'-11"

℄ SILL
℄ PLATE
AXIS OF MOMENTS, ST4 WF12

4'-0" | 4'-0" | 4'-0" | 4'-0"

16'-0"

## TYPICAL CROSS-SECTION
### SCALE: ¾" = 1'-0"

Exhibit 2

| 3 | 2 | 1 |

2'-0"  5'-6"

7U98 SILL SPLICERS

12'-0"    11'-6"

6"x6" LAM TIMBER SILL

8'-0"    11'-6"

E PLANS FOR LENGTHS OF MEMBERS

-1¾"Ø TIE BETWEEN SILLS

"6"x3-0"
)" RIDGE BEAM
ST4 WF 2 @ 4'-0"OC
8x8x1 PL    L4x3x½

10WF25   1½"Ø   4'-0"  4'-0"

1½"Ø  4'-0"  4'-0"  10'-0"

10WF25  1½"Ø

4'-0"  4'-0"

32-0"    4'-0" 1'-6" 5'-0"  1-1½"

4'-1½"

RAMING PLAN
0'

-2"x10" RIDGE BEAM
- 8"x8" x 1" PLATE
- ½"Ø x 4½" BOLTS

10 WF 25

8'-0" 2

6½"x8x1 PLATE
7U98
1"Ø BOLTS, 1'-0"O.C
2-6"x6"

4'-0"    4'-0"

SS-SECTION
"-1'-0"

| APPLICATION | | UNLESS OTHERWISE SPECIFIED | | U. S. ARMY ENGINEER RESEARCH AND DEVELOPMENT LABORATORIES CORPS OF ENGINEERS FORT BELVOIR, VA |
| NEXT ASSY | USED ON | DIMENSIONS ARE IN INCHES TOLERANCES ON FRACTIONS DECIMALS ANGLES | | |
| | | BREAK SHARP EDGES TO ALL FILLETS TO | | CIVILIAN GROUP SHELTER |
| | | MATERIAL | | METAL - TIMBER |

CODE IDENT. NO. | SIZE
97403 | D | 13208E5679

SCALE: AS SHOWN | SHEET 1 OF 2

| 5 | 4 | 3 |

**D**

$7\frac{1}{4}"$

$4\frac{1}{2}"$

$1\frac{1}{2}"$ TIE ROD

$1"\phi$ TIE BOLT

$\frac{1}{2}" \times 4"$ LAG SCREW

7 ⊔ 98

$6\frac{1}{2} \times 8 \times 1$ PLATE

$12\frac{1}{2}"$

$\angle 4 \times 3 \times \frac{1}{2}$

DETAIL "A"

**C**

$4\frac{1}{4} \times 13$

$2\frac{1}{4}"$

$4"$

$\frac{1}{2}$ PLATE
THREADED SOCKET

10 W 25

$\frac{1}{2}\phi$ BOLTS

$1\frac{1}{2}"\phi$ UPSET
SCREW END

$\frac{1}{4}"$ PLATE

$6\frac{1}{2}"$

DETAIL "B"

DETAIL "D"

**B**

$\frac{1}{2}"\phi$ BOLTS

THREADED SOCKET

$1\frac{1}{2}"\phi$ UPSET SCREW END

10 W 25

"E"

"E"

$\frac{1}{2}"$ PLATES

DETAIL "C"

**A**

$1"$ | $5"$ | $5"$

$1\frac{1}{2}"$

$7"$

$5"$

$1\frac{1}{2}"$

$6"$

$1"$ PLATE

$\frac{1}{2}"$ PLATE

$2\frac{7}{8}"$

$10\frac{3}{8}"$

SECTION "E-E"

| 5 | 4 | 3 |

D

1" φ TIE BOLT

- 7⌐98

1/2" x 4" LAG SCREW

6 1/2 x 8 x 1 PLATE



∠ 4 x 3 x 1/2

C

10 ⱲF 25

> 1 1/2" φ BOLTS

"D"

10 ⱲF 25

"E"

∅ — 1/2 ▷

1" PLATE

— 1 1/2" PLATE

B

A

APPENDIX E                                Exhibit 1

CONCRETE AND MASONRY STRUCTURES

by

Stanley E. Woell

1.  General. The basic design considerations for the shelter constructed with concrete material are the same as with other shelters in this report. In consideration of designs for the shelter, the research material published by Dr. Nathan M. Newmark was constantly used for reference and guidance.

2.  Design Type. Various designs of concrete and masonry shelters were considered. All designs were primarily evaluated on the basic requirement of the project (i. e., that the shelter would be erected by an inexperienced group of people with practically no equipment). This basic requirement has circumscribed design selection. After an initial consideration, the use of masonry construction was eliminated because of the skill involved. Also, the mixing and application of the wet concrete on site for the complete structure was discarded for the same reason. This left only precast concrete materials that could be considered in the structural design. Because of envisaged lack of erection equipment, such precast concrete units should be manhandable. A fully precast concrete structure would have units too heavy to be manhandable and would require erection skill beyond the capabilities of inexperienced people. This implied the necessity of including in the design a structural frame that could be erected readily without the use of heavy equipment and which would support the precast concrete units.

The design consists of a steel frame utilizing 8 WF 20 members for columns and beams and "Rackle-Lite" precast slabs. All of the units of the design will be manhandable, the largest unit requiring not more than four men for handling. Aluminum could be used in the framework instead of steel which would make individual units materially lighter. The design is for a rectangular structure to accommodate 60 persons. Dimensions are 10 by 60 by 7 feet. This gives 10 square feet of floor space and 70 cubic feet of air volume per person.

3.  Design Pressures. The structure has been designed for a static loading of 23.5 psi. This static loading design includes 3.5 psi for a maximum soil coverage of 5 feet on the structure and

a potential blast loading of 20 psi from a megaton weapon. Dr. Newmark gives a 0.5 lateral load factor which has been used in the design, giving a 10-psi lateral overpressure loading. The maximum depth at which the shelter excavations will be made has been assumed to be about 13.5 feet. For a cohesionless soil, the side pressure due to the weight of the earth at 13.5 feet below ground surface is 3.66 psi. A loading of 3.5 psi was then added to the 10-psi lateral overpressure giving a total design lateral loading of 13.5 psi.

The side design load is assumed to act uniformly in all directions at 13.5 psi at all levels between 5 and 13 feet below grade. This is not an accurate assumption, but it is felt that since the overpressure is so arbitrary and there is little knowledge of the actual effects of side pressure as a function of depth, this assumption could be made without weakening the structure.

4. Bulkheads. Due to the 4- by 6-foot opening in one bulkhead (called entrance bulkhead), the design of the bulkheads posed a unique problem. As discussed above, the side pressure was assumed to act uniformly at depths between 5 and 13 feet. With an opening in the entrance bulkhead, the areas of the two bulkheads were not the same. Therefore, there is an unbalance in forces if the forces are transmitted longitudinally through the shelter and must be taken up by the soil. To transmit the forces longitudinally through the shelter, each bulkhead had to be designed so that the forces transmitted through the braces from each end had the same ratio. After several different attempts to achieve such a design, it was found that it was impossible to have a simple design with members less than 200 pounds.

Another design investigated was a triangular frame where the hypotenuse was exposed to the earth and the two legs of the triangle were the supports withstanding the load. Again, members under 200 pounds could not be used.

The final design uses the principle of a deadman. Two tie rods are connected to each column which lead to the deadmen. The tie rods are so connected to the columns to give each a zero moment at its center. The deadmen have been placed approximately 8 feet away from the bulkheads and 14 feet below grade. In this position, they have more resistance to the loading and during construction, the earth will provide the forms necessary to construct the deadmen.

5. Design Data. The computations from which the design of the shelter resulted are found in Appendix G. As stated earlier, the uniform loads used in designing the shelter were 23.5-psi

vertical loading and 13.5-psi horizontal loading. Since in an atomic blast, the shock front hits an object instantaneously with no build-up time, and remains at its peak overpressure only a maximum of a few milliseconds with a gradual decay, the load is treated essentially as a dynamic loading initially with an ensuing static loading. Also, the plastic method of design is used with design stresses of materials as follows:

a.   Wood - Southern Pine Timber - Dense Structural or Douglas Fir Timber - B. & S. Dense Construction.

   (1)   640 psi - horizontal shear.

   (2)   1,100 psi - compression ⊥ to grain.

   (3)   4,800 psi - compression to grain.

   (4)   6,000 psi - bending.

b.   A-36 Structural Steel.

   (1)   50,000 psi - direct compression.

   (2)   50,000 psi - bending.

c.   Deadman Concrete.   3,000 psi (28-day strength) - compression.

d.   Rackle-Lite.   600 psi (28-day strength) - compression. Reinforced to carry an ultimate load of 240 psf.

e.   Soil.

   (1)   2,000 psf - backfill soil compacted.

   (2)   3,000 psf - undisturbed soil.

In the design of steel frame for the shelter, rigid connections are used wherever possible to reduce the size of the members. It will be noted that the beams supporting the roof are rigidly connected to the columns, but the separator or brace between the columns is simply connected. By simply connecting the brace, there is no moment transmitted to it thus reducing its size.

The beams used to support the roof on both bulkheads are angles. Their purpose is to stabilize the planking so that no longitudinal motion will occur when backfilling. Likewise, the

bracing around the top of the main shelter consists of angles which prevent transverse motion of the planking.

The main problem in designing the entranceway was an unbalance of forces due to the opening into the shelter. Since the design assumes 13.5 psi acting uniformly on the walls and the pressure inside the shelter is ambient, there is an unbalance of 46,680 pounds. To counteract this unbalance, it was first determined whether the frictional force of the soil on the concrete planking would provide the needed resistance. It was found that a possible friction force of 61,670 pounds would act if motion were impending. Since this force is greater than the unbalance, there should not be any movement of the entranceway into the shelter as long as the roof planks are fastened securely to the framework. To be certain that the friction forces act as calculated, angles are fastened to the end beams holding any plank in place that slips on the frame.

The problem in designing the blast cover for the entranceway was the negative pressure phase effect on the cover. For the cover to remain intact, its weight would have to be equal to the negative pressure of 2.6 psi times the area of the cover or approximately 9,360 pounds. This was definitely out of the question. The next best thing is a deadman connected to each footing. This is the design incorporated here.

In designing the blast cover, it has been assumed that the fasteners holding the cover to the footings are secure enough to consider the planks in the cover as rigidly connected at both ends. This decreases the weight of the cover considerably, but even with this simplification the cover is too heavy to be handled as one unit.

6. **Cost Analysis.** The cost analysis represents the cost of materials only.

Table XI is a list of the materials required to build the shelter. The materials are listed under three divisions: main shelter, entranceway, and miscellaneous. The source, weight, unit cost, and total cost are given for each item.

The unit cost of the steel members is for A-36 structural steel and includes the expense of precutting all items to the correct length at the factory. The price does not include pre-drilling of holes for joints or transportation to the shelter site. To estimate the cost of timber, the unit price of $260/1,000 fbm has been used for members which will be in contact with the soil. For timber not in contact with the soil, the unit price of $225/1,000 fbm has been used. The extra $35/1,000 fbm for members contacting soil is

Table XI. Bill of Materials - Concrete Structure

(a) Main Shelter

| Use | Quantity | Description | Source* | Total Weight (lb) | Unit Cost ($) | Total Cost ($) |
|---|---|---|---|---|---|---|
| Side & Roof | | | | | | |
| Beam | 29 | 8 WF 20 12' Lg | 3 | 6960.0 | .067/lb | 467.00 |
| Column | 58 | 8 WF 20 7'-4" Lg | 3 | 8502.8 | .067/lb | 570.00 |
| Brace | 60 | L 5 x 3-1/2 x 1/4  2' Lg | 1 | 340.0 | .0650/lb | 54.60 |
| Brace | 29 | L 1-1/2 x 1-1/2 x 3/16 10'-8" Lg | 2 | 458.2 | .0775/lb | 35.60 |
| Brace | 60 | L 1 x 1 x 1/8  2' Lg | 2 | 96.0 | .0835/lb | 8.02 |
| Bulk. Enc. | | | | | | |
| | 2 | Bar 1" φ  10'-8" Lg | 2 | 56.8 | .0745/lb | 4.24 |
| | 3 | Bar 1" φ  10'-4" Lg | 2 | 82.8 | .0745/lb | 6.18 |
| | 2 | Bar 1" φ  8'-5" Lg | 2 | 45.0 | .0745/lb | 3.36 |
| | 3 | Bar 1" φ  8'-2-1/2" Lg | 2 | 65.4 | .0745/lb | 4.88 |
| | 10 | No. 3-1/2 Clevis | 2 | 60.0 | 1.00/Clevis | 10.00 |
| | 10 | 1-1/2" Dia. Pin 2" Lg | 2 | 5.0 | .20/Pin | 2.00 |
| | 10 | 6" Lg 5.25 lb Turnbuckle | 1 | 52.5 | 1.00/Turnbk | 10.00 |
| | 10 | ST 5 I 12.7 6" Lg | 2 | 63.5 | .0795/lb | 5.05 |
| Beam | 1 | L 5 x 3-1/2 x 5/8  12' Lg | 2 | 201.6 | .0650/lb | 13.10 |
| Column | 2 | 5 WF 16  8' Lg | 3 | 256.0 | .0680/lb | 17.40 |
| Column | 3 | 5 I 10  8' Lg | 2 | 240.0 | .0690/lb | 16.55 |
| Column | 4 | L 1 x 1 x 3/4  7'-6" Lg | 2 | 34.8 | .0835/lb | 2.91 |
| Brace | 2 | L 1 x 1 x 1/8  2'-10" Lg | 2 | 4.5 | .0835/lb | .38 |
| Brace | 2 | L 1 x 1 x 1/8  2'-8" Lg | 2 | 4.3 | .0835/lb | .36 |
| Bulk. Ent. | | | | | | |
| | 2 | Bar 3/4" φ  12'-3" Lg | 2 | 36.8 | .0745/lb | 2.74 |
| | 2 | Bar 3/4" φ  12'-1" Lg | 2 | 36.2 | .0745/lb | 2.70 |
| | 2 | Bar 3/4" φ  11'-10" Lg | 2 | 35.5 | .0745/lb | 2.65 |

Table XI (cont'd)

| Use | Quantity | Description | Source* | Total Weight (lb) | Unit Cost ($) | Total Cost ($) |
|---|---|---|---|---|---|---|
| Bulk. Ent. | 2 | Bar 3/4" ⌀ 9'-10" Lg | 2 | 29.5 | .0745/lb | 2.20 |
| | 2 | Bar 3/4" ⌀ 9'-7" Lg | 2 | 28.8 | .0745/lb | 2.15 |
| | 2 | Bar 3/4" ⌀ 9'-5" Lg | 2 | 28.2 | .0745/lb | 2.10 |
| | 12 | ST 4 I 9.2 6" Lg | 2 | 55.2 | .805/lb | 4.45 |
| | 12 | No. 3 Clevis | 2 | 48.0 | 1.00/Clevis | 12.00 |
| | 12 | 1-1/2" Dia. Pin 2" Lg | 2 | 6.0 | .20/Pin | 2.40 |
| | 12 | 6" Lg 2.72 lb Turnbuckle | 2 | 32.6 | 1.00/Turnbk | 12.00 |
| Beam | 1 | L 6 x 4 x 5/16 12' Lg | 2 | 123.6 | .0700/lb | 8.65 |
| Beam | 1 | L 6 x 4 x 5/16 4'-6" Lg | 2 | 54.0 | .0700/lb | 3.78 |
| Column | 2 | 10 B 15 8' Lg | 3 | 240.0 | .0690/lb | 16.60 |
| Column | 4 | I 7.7 8' Lg | 2 | 246.4 | .0710/lb | 17.50 |
| Braces | 4 | L 1-1/2 x 1-1/2 x 3/16 8'-0" Lg | 2 | 57.6 | .0775/lb | 4.47 |
| Brace | 2 | L 1 x 1 x 1/8 1'-8" Lg | 2 | 2.7 | .0835/lb | .23 |
| Brace | 2 | L 1 x 1 x 1/8 1'-3" Lg | 2 | 2.0 | .0835/lb | .17 |
| Covering | | | | | | |
| Roof | 3 | 4" x 24" x 6'-2" Rackle-Lite | 4 | 555.0 | .57/sq ft | 7.05 |
| Roof | 3 | 4" x 24" x 6'-1" Rackle-Lite | 4 | 547.5 | .57/sq ft | 6.95 |
| Roof | 55 | 4" x 24" x 6'-0" Rackle-Lite | 4 | 9900.0 | .57/sq ft | 125.50 |
| Blk Enc | 2 | 4" x 24" x 5'-9" Rackle-Lite | 4 | 345.0 | .57/sq ft | 4.38 |
| Roof | 3 | 4" x 24" x 4'-2" Rackle-Lite | 4 | 375.0 | .57/sq ft | 4.76 |
| Blk Enc | 4 | 4" x 24" x 3'-1" Rackle-Lite | 4 | 370.0 | .57/sq ft | 4.69 |
| Roof | 3 | 4" x 24" x 2'-1" Rackle-Lite | 4 | 187.5 | .57/sq ft | 2.39 |
| Sides | 8 | 3" x 24" x 6'-3" Rackle-Lite | 4 | 1200.0 | .55/sq ft | 13.75 |
| Sides | 68 | 3" x 24" x 6'-0" Rackle-Lite | 4 | 9792.0 | .55/sq ft | 112.20 |
| Sides | 4 | 3" x 24" x 4'-3" Rackle-Lite | 4 | 408.0 | .55/sq ft | 4.68 |
| Blk Ent | 8 | 3" x 24" x 3'-10" Rackle-Lite | 4 | 726.9 | .55/sq ft | 8.42 |
| Sides | 4 | 3" x 24" x 2'-3" Rackle-Lite | 4 | 216.0 | .55/sq ft | 2.48 |

Table XI (cont'd)

| Use | Quantity | Description | Source* | Total Weight (lb) | Unit Cost ($) | Total Cost ($) |
|---|---|---|---|---|---|---|
| Blk. Ent. | 1 | 18" x 5" x 13' Timber | 1 | 248.3 | 260/1000 BM | 25.35 |
| Blk. Enc. | 1 | 16" x 5" x 13" Timber | 1 | 221.0 | 260/1000 BM | 23.06 |
| Sides | 4 | 14" x 3" x 8' Timber | 1 | 275.2 | 260/1000 BM | 29.12 |
| Sides | 14 | 14" x 3" x 6' Timber | 1 | 722.4 | 260/1000 BM | 76.44 |
| Blk Ent | 1 | 12" x 3" x 4'-6" Timber | 1 | 32.9 | 225/1000 BM | 3.04 |
| Blk Ent | 1 | 9" x 3" x 4'-6" Timber | 1 | 24.3 | 260/1000 BM | 2.63 |
| | 300 | 3-1/2" Spikes | 1 | 9.0 | 2.00/100 | 6.00 |
| Blk Ent | 4 | 1'-9" x 1/4" Tie Rods | 2 | 1.2 | .40/Rod | 1.60 |
| Sides | 64 | 1'-5" x 1/4" Tie Rods | 2 | 15.2 | .38/Rod | 24.32 |
| Blk Ent | 2 | 11" x 5" x 1/4" PL | 2 | 7.8 | .0850/lb | .66 |
| Sides | 58 | 10" x 8" x 1/4" PL | 2 | 330.6 | .0850/lb | 28.10 |
| Blk Enc | 4 | 7" x 6" x 1/4" PL | 2 | 12.0 | .0850/lb | 1.02 |
| Blk Enc | 2 | 6" x 6" x 1/4" PL | 2 | 5.2 | .0850/lb | .44 |
| Blk Ent | 4 | 6" x 5" x 1/4" PL | 2 | 8.4 | .0850/lb | .72 |
| Blk Ent | 2 | 6  8.2  4.5' Lg | 2 | 73.8 | .0695/lb | 5.15 |
| Sides | 32 | 5.4  6' Lg | 2 | 1036.8 | .0710/lb | 73.50 |
| Deadman | 60 | 96 lb Sand Gravel Sakrete | 5 | 5760.0 | 155 | 93.00 |
| | 16 | No. 4 Bars 12'-6" Lg | 2 | 133.6 | .0300/lb | 10.70 |
| | 24 | No. 3 Ties 3'-6" Lg | 2 | 31.6 | .0870/lb | 2.75 |
| | | (b) Entranceway | | | | |
| Footings | 1 | 14" x 4" x 3'-10" Timber | 1 | 45.6 | 260/1000 BM | 4.65 |
| | 2 | 14" x 3" x 4' Timber | 1 | 68.8 | 260/1000 BM | 7.28 |
| | 1 | 12" x 3" x 4' Timber | 1 | 29.2 | 260/1000 BM | 3.12 |
| | 2 | 8" x 3" x 3'-10" Timber | 1 | 36.8 | 260/1000 BM | 3.98 |
| | 1 | 8" x 3" x 3'-1" Timber | 1 | 14.8 | 260/1000 BM | 1.56 |

Table XI (cont'd)

| Use | Quantity | Description | Source* | Total Weight (lb) | Unit Cost ($) | Total Cost ($) |
|---|---|---|---|---|---|---|
| Footings | 1 | 6" x 3" x 2'-2" Timber | 1 | 8.9 | 260/1000 BM | .85 |
| | 2 | 6" x 2" x 2'-1" Timber | 1 | 9.2 | 260/1000 BM | 1.04 |
| | 96 | 3-1/2" Spikes | 1 | 3.0 | 2.00/100 | 2.00 |
| | 1 | 6" x 6" x 1/4" PL | 2 | 2.6 | .0850/lb | .22 |
| | 3 | 5" x 5" x 1/4" PL | 2 | 5.4 | .0850/lb | .46 |
| | 10 | 5" x 4" x 1/4" PL | 2 | 14.0 | .0850/lb | 1.19 |
| | 10 | 4" x 4" x 1/4" PL | 2 | 11.0 | .0850/lb | .94 |
| Framing | 1 | 5 WF 16 6'-0" Lg | 3 | 96.0 | .0680/lb | 6.55 |
| | 6 | 5 WF 16 5'-9" Lg | 3 | 552.0 | .0680/lb | 37.50 |
| | 6 | 5 I 10 3'-10" Lg | 2 | 230.0 | .0690/lb | 15.88 |
| | 4 | 5 I 10 2'-6" Lg | 2 | 100.0 | .0690/lb | 6.90 |
| | 1 | 4 WF 10 6'-0" Lg | 6 | 60.0 | .0690/lb | 4.14 |
| | 2 | 4 WF 10 5'-10" Lg | 6 | 116.7 | .0690/lb | 8.04 |
| | 6 | 4 WF 10 5'-9" Lg | 6 | 345.0 | .0690/lb | 23.80 |
| | 1 | 4 WF 10 5'-8" Lg | 6 | 56.7 | .0690/lb | 3.91 |
| | 1 | 4 WF 10 3'-8" Lg | 6 | 36.7 | .0690/lb | 2.53 |
| | 8 | 4 I 9.5 5'-8" Lg | 2 | 430.2 | .0710/lb | 31.00 |
| | 1 | 4 I 9.5 3'-10" Lg | 2 | 36.4 | .0710/lb | 2.58 |
| | 3 | 4 I 9.5 3'-8" Lg | 2 | 104.5 | .0710/lb | 7.42 |
| | 1 | 4 I 7.7 6'-0" Lg | 2 | 46.2 | .0710/lb | 3.28 |
| | 2 | 4 I 7.7 5'-10" Lg | 2 | 79.8 | .0710/lb | 5.67 |
| | 1 | 4 I 7.7 5'-8" Lg | 2 | 43.6 | .0710/lb | 3.10 |
| | 1 | 4 I 7.7 3'-6" Lg | 2 | 27.0 | .0710/lb | 1.92 |
| | 2 | 4 I 7.7 3'-5" Lg | 2 | 52.6 | .0710/lb | 3.74 |
| | 1 | 4 I 7.7 3'-4" Lg | 2 | 25.7 | .0710/lb | 1.83 |
| | 2 | 4 I 7.7 3'-1" Lg | 2 | 47.4 | .0710/lb | 3.36 |
| | 6 | 3 I 5.7 5'-9" Lg | 2 | 203.7 | .0730/lb | 14.90 |

**FOUNDATION & FOOTING PLAN**
SCALE: $\frac{1}{4}$"=1'-0"

76'-6"

8'-9" | 1'-0" | 8'-0"

7 - 5"x14" @ 6'-0" LG = 42'-0"

8 SPLICES - 4 ⌊ 5.4 @ 6'-0" = 48'-0"

29 - 8"x10" x $\frac{1}{4}$" PL @ 2'-0" C.C = 56'-0"

5'-0"

1'-0"

5"x11"x$\frac{1}{4}$" PL

3"x14"

$\frac{1}{2}$"x17" TIE RODS (SPACE ACCORDING TO DETAIL "A")

8'-0"

5'

6"x14"

DETAIL "A"

#4 REINF BARS,
#3 TIES, 12" O.C
(SEE DETAIL)

5"x6"x$\frac{1}{4}$" PL

6 ⌊ 8.2 (BOTH SIDES)
$\frac{1}{2}$"x21" TIE RODS

5"x6"x$\frac{1}{4}$" PL

12'-6"

11'-4"

5"x16"
TIMBER

6"x7"x$\frac{1}{2}$" PL

18" DEEP CONC
DEADMAN
(EARTH FORM)

5"x11"x$\frac{1}{4}$" PL

5"x18" TIMBER SILL

SPLICED TIMBER SILL (SAME AS OPPOSITE SIDE)

6"x6"x

ENTRANCE END

---

**FLOOR PLAN**
SCALE: $\frac{1}{4}$"=1'-0"

⌊ 1$\frac{1}{2}$ x 1$\frac{1}{2}$ x $\frac{5}{16}$

10B15

60'-6"

60'-0"

29 - 8 WF 20 @ 2'-0" O.C = 56'-0"

3" THICK "RACKLE-LITE" ROOF PLANKS

2'-0"

C6
C5
C4

⌊ I 7.7

C3

C2

C1

BOTTOM SPREADERS - ⌊ 1$\frac{1}{2}$ x 1$\frac{1}{2}$ x $\frac{5}{16}$
TOP BEAMS 8 WF 20

51

3" THICK "RACKLE-LITE"

D

'6'-6"

'-0" LG = 42'-0"

6'-0" = 48'-0"

2'-0" C.C = 56'-0"

NG TO DETAIL "A"

DETAIL "A"

8'-0"

5'-0"

1'-0"

6"x14"

5"x16"
TIMBER SILL

6"x7"x¼" PL

AS OPPOSITE SIDE)

6"x6"x¼" PL

7'-9"

1'-6"

6"x6"x¼" PL

CONC DEADMAN
(IDENT TO OTHER)

12'-6"

SECTION "B-B"
SCALE: 1½"=1'-0"

GL 18.2 x 4'-6"

¼" x 21"
TIE RODS

5"x18"x 12'-6"

C

OTING PLAN
-0"

60'-6"

60'-0"

20 @ 2'-0" O.C.= 56'-0"

3" THICK "RACKLE-LITE" ROOF PLANKS

¼"x 17" TIE ROD

4 ⌐ 5.4

5"x14"

¼"x17" TIE RODS

6'-0"

SPLICE

3'-0"

3'-0"

4 ⌐ 5.4

5"x14"

5"x14"

1'-0"

1'-6"

1'-0"

1'-6"

1'-0"

DETAIL "A"
SILL SPLICING DETAIL
SCALE: 1½"=1'-0"

SPREADERS - ∠ 1½"x1½"x⅜
BEAMS 8 WF 20

5 WF 16

∠ 1x1x³⁄₁₆

5I 10

C11

C10

C9

C8

C7

12'-0"

1'-6"

3" 4" 4" 4" 3"

TIE DOWN RODS

#4 BARS SPACED AS SHOWN

#3 TIES, 12" O.C.

4" THICK "RACKLE-LITE"
ROOF PLANKS

AN
-0"

SECTION "C-C"
SCALE: 1½"=1'-0"

3      2      1

5      4      3

∠5×3½×¼

1-10B15,
2-∠1½×1½×³⁄₁₆

∠6×4-⁵⁄₁₆×2'0

1-10B15, 2-∠1½×1½×³⁄₁₆

D

3"×12" TIMBER
∠6×4×⁵⁄₁₆×6"
(BEHIND)

4177

4177

3'-0"

3"×9" TIMBER

∠1×1×⅛

∠1×1×⅛

½" PLATES PREWELDED
TO VERTICAL MEMBERS
(SEE FOUNDATION PLAN)

**FRAMING ELEVATION
ENTRANCE BULKHEAD**
SCALE ½"=1'-0"

BACK FILL
"VISQUEEN" PLASTIC WATERPROOFING
4" THICK "RACKLE-LITE"

∠5×3½×¼

8 W 20×12'-0"

3" THICK
"RACKLE-LITE"

8W20

8W2

∠1½×1½×³⁄₁₆

½" PLATE

**TYPICAL CROSS-SECT[ION]**
SCALE ½"=1'-0"

28-∠5×3½×¼×1'-11½"

2'-2"

∠6×4×⁵⁄₁₆

∠5×3½×¼

2'-2¼"

∠5×3½×¼

∠5×3½×⅜

DETAIL

G

G

∠1½×1½×³⁄₁₆

10 B15

3"×12" TIMBER

CUT & CHIP

DEADMAN TIEDOWNS
(SEE DETAIL DWG)

4177

3"×9"
TIMBER

5110

4"

∠5×3½×¼

∠6×4×⁵⁄₁₆

**SECTION "D-D"**
SCALE: 3"=1'-0"

TOP OF PLATES

2'-0"

∠1×1×⅛

1'-0"

**LONGITUDINAL SECTION ALONG ₵**
SCALE: ½"=1'-0"

BACKFILL
"VISQUEEN PLASTIC WATERPROOFING
4" THICK "KACKLE LITE"

∠ 5 × 3½ × ¼

8 W 20 × 2'-0"

THICK
"KACKLE LITE"

8 W 20     8 W 20

∠ 1½ × 1½ × ³⁄₁₆

ED
BERS
PLAN)

½" PLATE

**TYPICAL CROSS-SECTION**
SCALE: ½"=1'-0"

∠ 5 × 3½ × ⅜ × 12'-0"     ∠ 5 × 3½ × ¼

1-5 W 16,
2-∠ 1 × 1 × ³⁄₁₆

1-5 W 16,
2-∠ 1 × 1 × ³⁄₁₆

E   E

5 I 10

½" PLATE     ∠ 1 × 1 × ⅛

**FRAMING ELEVATION
ENCLOSED BULKHEAD**
SCALE: ½"=1'-0"

2'-2½"

∠ 5 × 3½ × ¼

4"

10 B 15

CUT & CHIP

∠ 6 × 4 × ³⁄₁₆

5 I 10

∠ 5 × 3½ × ¼

**SECTION "D-D"**
SCALE: 3"=1'-0"

G

∠ 5 × 3½ × ¼     ∠ 5 × 3½ × ¼

∠ 5 × 3½ × ¼

8 W 20     8 W 20

G

8 W 20

**DETAIL "F"**
SCALE: 3"=1'-0"

**SECTION "G-G"**
SCALE: 3"=1'-0"

∠ 1 × 1 × ³⁄₁₆

8 W 16

∠ 5 × 3½ × ⅜

CUT & CHIP

3½"

∠ 5 × 3½ × ¼

**SECTION "E-E"**
SCALE: 3"=1'-0"

| | | |
|---|---|---|
| **5** | **4** | **3** |

ENTRANCE BULKHEAD

ROOF

ENCLOSED BULKHEAD

SIDE
(OTHER SIDE IDENT.)

## "RACKLE-LITE" ROOF PLANK PLACEMENT PATTERNS
SCALE: $\frac{1}{8}$"=1'-0"

| SYMBOL | SIZE |
|---|---|
| (A) | 6'-3" x 2'-0" x 3 |
| (B) | 6'-0" x 2'-0" x 3 |
| (C) | 2'-3" x 2'-0" x ? |
| (D) | 4'-3" x 2'-0" x ? |
| (E) | 6'-1" x 2'-0" x 4 |
| (F) | 2'-1" x 2'-0" x 4 |
| (G) | 6'-0" x 2'-0" x 4 |
| (H) | 6'-2" x 2'-0" x ? |
| (I) | 4'-2" x 2'-0" x 4 |
| (J) | 3'-10" x 2'-0" x ? |
| (K) | 3'-1" x 2'-0" x 4 |
| (L) | 5'-9" x 2'-0" x 4 |

NOTE: ALL JOINTS TO BE SEALED WITH

STRUCTURAL TEE
CLEVIS & PIN
ROUND ROD
TURNBUCKLE
₵ DEADMAN
2" RADIUS (TYPICAL)

TOP OF PLATE
₵ DEADMAN

METHOD I          METHOD II

## DEADMAN TIEDOWNS
SCALE: $\frac{3}{4}$"=1'-0"

| COL DESIG (SEE FL PL) | CONST METHOD | DIME A | B | |
|---|---|---|---|---|
| C1 | II | 8'-0" | 2'-0" | 1' |
| C2 | I | 8'-0" | 2'-0" | 1' |
| C3 | I | 8'-0" | 2'-0" | 1' |
| C4 | I | 8'-0" | 2'-0" | 1' |
| C5 | I | 8'-0" | 2'-0" | 1' |
| C6 | II | 8'-0" | 2'-0" | 1' |
| C7 | II | 7'-0" | 1'-0" | 1' |
| C8 | I | 7'-0" | 1'-0" | 1' |
| C9 | I | 7'-0" | 1'-0" | 1' |
| C10 | I | 7'-0" | 1'-0" | 1' |
| C11 | II | 7'-0" | 1'-0" | 1' |

NOTES:
1. TIE DOWN LENGTH IS
   CLEVIS, TURNBUCKLE

2. ST 5 12.7 FOR COLS. C

| | | |
|---|---|---|
| **5** | **4** | **3** |

| 3 | 2 | 1 |

ENCLOSED BULKHEAD

| SYMBOL | SIZE | QTY REQ'D |
|---|---|---|
| (A) | 6'-3" x 2'-0" x 3" | 8 |
| (B) | 6'-0" x 2'-0" x 3" | 68 |
| (C) | 2'-5" x 2'-0" x 3" | 4 |
| (D) | 4'-3" x 2'-0" x 3" | 4 |
| (E) | 6'-1" x 2'-0" x 4" | 3 |
| (F) | 2'-1" x 2'-0" x 4" | 3 |
| (G) | 6'-0" x 2'-0" x 4" | 55 |
| (H) | 6'-2" x 2'-0" x 4" | 3 |
| (I) | 4'-2" x 2'-0" x 4" | 3 |
| (J) | 3'-10" x 2'-0" x 3" | 8 |
| (K) | 3'-1" x 2'-0" x 4" | 4 |
| (L) | 5'-9" x 2'-0" x 4" | 2 |

NOTE   ALL JOINTS TO BE SEALED WITH ASPHALTIC CEMENT

| COL DESIG (SEE FL PL) | CONST METHOD | DIMENSIONS | | | | | STRUCT TEE 6" LONG | CLEVIS NO | PIN DIA(IN) | ROD DIA(IN) | TIE DOWN LENGTH | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | A | B | C | D | E | | | | | UPPER | LOWER |
| C1 | II | 8'-0" | 2'-0" | 1'-11" | 3'-10½" | 2" | ST4I92 | 3 | 1½ | ¾ | 12'-1" | 9'-10" |
| C2 | I | 8'-0" | 2'-0" | 1'-10" | 3'-10" | 2" | ST4I92 | 3 | 1½ | ¾ | 11'-10" | 9'-7" |
| C3 | I | 8'-0" | 2'-0" | 1'-4" | 4'-10" | 2" | ST4I92 | 3 | 1½ | ¾ | 12'-5" | 9'-5" |
| C4 | I | 8'-0" | 2'-0" | 1'-4" | 4'-10" | 2" | ST4I92 | 3 | 1½ | ¾ | 12'-5" | 9'-5" |
| C5 | I | 8'-0" | 2'-0" | 1'-10" | 3'-10" | 2" | ST4I92 | 3 | 1½ | ¾ | 11'-10" | 9'-7" |
| C6 | II | 8'-0" | 2'-0" | 1'-11" | 3'-10½" | 2" | ST4I92 | 3 | 1½ | ¾ | 12'-1" | 9'-10" |
| C7 | II | 7'-0" | 1'-0" | 1'-11" | 3'-10½" | 2¼" | ST5I127 | 3½ | 1½ | 1 | 10'-8" | 8'-5" |
| C8 | I | 7'-0" | 1'-0" | 1'-10" | 3'-9½" | 2¼" | ST5I127 | 3½ | 1½ | 1 | 10'-4" | 8'-2½" |
| C9 | I | 7'-0" | 1'-0" | 1'-10" | 3'-9½" | 2¼" | ST5I127 | 3½ | 1½ | 1 | 10'-4" | 8'-2½" |
| C10 | I | 7'-0" | 1'-0" | 1'-10" | 3'-9½" | 2¼" | ST5I127 | 3½ | 1½ | 1 | 10'-4" | 8'-2½" |
| C11 | II | 7'-0" | 1'-0" | 1'-11" | 3'-10½" | 2¼" | ST5I127 | 3½ | 1½ | 1 | 10'-8" | 8'-5" |

NOTES:
1. TIE DOWN LENGTH IS MEASURED FROM ℄ PIN AND INCLUDES LENGTH OF CLEVIS, TURNBUCKLE AND BEND OF ROD

2 ST5I27 FOR COLS C7 & C11 MUST BE REDUCED IN WIDTH TO FIT SWFIG

2

| APPLICATION | | UNLESS OTHERWISE SPECIFIED | | U. S. ARMY ENGINEER RESEARCH AND DEVELOPMENT LABORATORIES   CORPS OF ENGINEERS FORT BELVOIR, VA |
|---|---|---|---|---|
| NEXT ASSY | USED ON | DIMENSIONS ARE IN INCHES TOLERANCES ON FRACTIONS   DECIMALS   ANGLES | RECOMMENDED: | |
| | | BREAK SHARP EDGES   TO ALL FILLETS   TO | REVIEWED FOR PRODUCTION: | CIVILIAN GROUP SHELTER PRECAST CONCRETE - METAL |
| | | MATERIAL | DRAWN BY   TRACED BY   CHECKED BY | |

CODE IDENT NO. 97403 | SIZE D | 13208 E 5647

SCALE: | SHEET 5 OF 7

| 5 | 4 | 3 |

DETAIL "A"
SCALE : 1" = 2"

ENTRANCE FLOOR
PLAN
SCALE : ¾" x 1'-0"

DETAIL "B"
TO BE FOR ALL 5 WF-16 COLUMNS
NOT OTHERWISE SPECIFIED
SCALE : 1" = 2"

SEE DETAIL "B" ①

FOOTING PLAN
SCALE: ¾" = 1'-0"
NOTE:    SEE - BILL OF MATERIALS

| 5 | 4 | 3 |

DETAIL "A"
SCALE: 1" = 2"

ENTRANCE FLOOR
PLAN
SCALE 3/4 x 1-0

DETAIL "B"
0 BE FOR ALL 5 WF-16 COLUMNS
NOT OTHERWISE SPECIFIED
SCALE : 1" = 2"

SEE DETAIL "B"

SEE DETAIL "A"
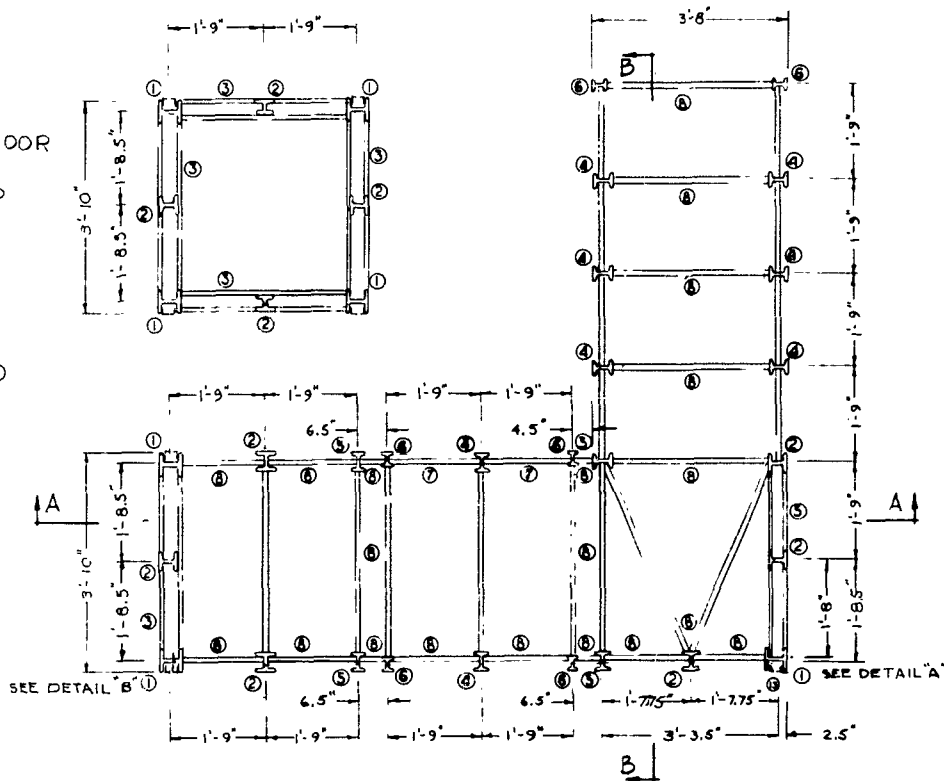
FLOOR PLAN
SCALE: 3/4 = 1'-0"
NOTE: SEE-BILL OF MATERIALS

CIVILIAN GROUP SHELTER
PRECAST CONCRETE-METAL

| APPLICATION | | | |
|---|---|---|---|
| NEXT ASSY | USED ON | | |

CODE IDENT. NO. 97403  D  13208E5647

SHEET 4 OF 7

1 |← 1'-9" →|← 1'-9" →|
|← 4'-6" (3×14) →|

2 |← 1'-9" →|
|← 2'-1" →|
(2×6)

3 |← 1'-9" →|← 1'9" →|
|← 3'-10" (3×8) →|

4 |← 3'-5" →|
|← 4'-0" (3×12) →|

5 |← 3'-4" →|
|← 3'-10" (4×14) →|

6 |← 1'.775" →|
|← 5" →|
|← 2'-4" →|
(3×6)

7 |← 1'-8" →|
|← 1'-4" →|
|← 3'-6" (3×8) →|

ENTRANCE ROOF
PLAN
SCALE : ¾" = 1'-0"

3'-10"

3'-10"

FOOTING DETAIL (BLOCKING)
SCALE: ¾" = 1'-0"

NOTE.

BILL OF MATERIAL

ENTRANCE ROOF
PLAN
SCALE : ¼"=1'-0"

ROOF PLAN
SCALE ¾"=1'-0"
NOTE: SEE - STEEL BILL OF MATERIAL

CIVILIAN GROUP SHELTER
PRECAST CONCRETE-METAL

| | | |
|---|---|---|
| APPLICATION | | |
| NEXT ASSY | USED ON | |

UNLESS OTHERWISE SPECIFIED
DIMENSIONS ARE IN INCHES
TOLERANCES ON
FRACTIONS   DECIMALS   ANGLES

BREAK SHARP EDGES       TO
ALL FILLETS          TO

MATERIAL

97403 **D** 13208 E5647

SHEET 5 OF 7

**5**      **4**      **3**

**D**

© G A
Ⓐ G © 
© G Ⓐ

WALL "A"
10'-9¾"

Ⓐ Ⓘ
Ⓘ Ⓐ
Ⓐ Ⓘ

WALL "B"
7'-2½"

Ⓛ Ⓙ
Ⓑ Ⓙ Ⓛ
Ⓛ Ⓙ

WALL "C"
11'-9½"

Ⓑ
Ⓑ
Ⓑ

**C**

Ⓔ
Ⓔ
Ⓔ

WALL "E"
3'-10"

Ⓕ
Ⓕ
Ⓕ

WALL "F"
3'-10"

Ⓔ
Ⓔ
Ⓔ

WALL "G"
3'-10"

Ⓔ
Ⓔ
Ⓔ

WALL "H"
3'-10"

Ⓔ
Ⓔ
Ⓔ

WALL "I"
3'-10"

PREC

SYMB
Ⓐ
Ⓑ
Ⓒ
Ⓓ
Ⓔ
Ⓕ
Ⓖ
Ⓗ
Ⓘ
Ⓙ
Ⓚ
Ⓛ
Ⓜ

WALL DETAILS
SCALE ⅜"=1'-0"

**B**

3'-9½"

Ⓓ Ⓓ
Ⓗ Ⓗ
Ⓗ Ⓗ

ENTRANCE ROOF
10'-10¾"

WALL "H"

WALL "C"

ROOF DETAILS
SCALE ¾"=1'-0"

**A**

ENTRANCE TO SHELTER
SCALE ¼"=1'-0"

D

WALL "C"
11'-9½

WALL "D"
7'-10½

WALL "I"
3'-10"

C

PRECAST CONCRETE SLABS

| SYMBOL | SIZE | QUAN. |
|--------|------|-------|
| A | 1-10" X 2-0" X 3" | 7 |
| B | 1-11" X 2-0" X 3" | 3 |
| C | 3-7" X 2-0" X 3" | 3 |
| D | 3-9" X 2-0 X 4" | 2 |
| E | 3-10" X 2-0" X 3" | 15 |
| F | 4-0" X 2-0" X 3" | 1 |
| G | 5-3" X 2-0" X 3" | 3 |
| H | 5-4" X 2-0" X 4" | 4 |
| I | 5-4" X 2-0" X 3" | 3 |
| J | 5-9" X 2-0" X 3" | 3 |
| K | 5-11 X 2-0" X 3" | 3 |
| L | 6-0 X 2-0 X 3" | 3 |

B

WALL "E" & "F"
WALL "I"
WALL "H"
WALL "D"
WALL "B"
WALL "C"
SHELTER WALL
WALL "A"

WALL LOCATOR

ENTRANCE TO SHELTER
SCALE ¼"=1'-0"

A

D

5'-6"

5'-0"

3"x3"x 2'-6"

5"x5"x 5'-0"

5"x8"x5'-0" (8 REQ'D)

4"x12"x 5'-6"

SEE NOTE 1

3"x 8"

3"x3"

5'-0"

3"x8"

4"x

WEDG

90°

4"x6"x 5'-6"

SEE NOTE 2

C

SEE NOTE 1

12'-0"

SEE NOTE 3

9'-0"

SEE NOTE 3

B

SEE NOTE 4

11'-9½"

SEE NOTE 3

10'-9¾"

## SECTION 'A'-A
SCALE ¾ = 1'-0"

## SECTION '
SCALE ¾

NOTES:
1. ROOF DETAIL SEE SHEET 5
2. WALL PLAN SEE SHEET 6
3. STEEL DETAIL SEE SHEET 4
4. FOOTING DETAIL SEE SHEET 5

SECTION B-B
SCALE 3/4"=1'-0"

BILL OF MATERIAL (STEEL)

| FIND NO. | DESCRIPTION |
|---|---|
| 1 | 5 WF-16 |
| 2 | 4 WF-10 |
| 3 | 5 I -10 |
| 4 | 4 I -9.5 |
| 5 | 4 I -7.7 |
| 6 | 3 I - 5.7 |
| 7 | L 1 X 1 X 1/8 |
| 8 | L 1 X 1 X 1/8 |
| 9 | ₽ 4 X 4 X 1/4 |
| 10 | ₽ 5 X 4 X 1/4 |
| 11 | R 5 X 5 X 1/4 |
| 12 | R 6 X 6 X 1/4 |
| 13 | L 4 X 3 X 1/4 |

CIVILIAN GROUP SHELTER
PRECAST CONCRETE- METAL

| CODE IDENT. NO. | SIZE | | |
|---|---|---|---|
| 97403 | D | 13208E 5647 | |

SHEET 7 OF 7

REVISIONS

| SYM | ZONE | DESCRIPTION | DATE | APPROVAL |
|---|---|---|---|---|
| | | | | |

APPENDIX F

PLASTIC STRUCTURES

by

John R. Fisher

1.   Plastic Materials as Structural Members.  The word
"plastic" is a nonspecific one which has come to comprise loosely
those primarily synthetic organic polymers which can be formed under
the influence of heat and pressure into comparatively permanent use-
ful shapes.  This large body of materials can be more exactly sub-
classified into two categories:  thermoplastic and thermosetting.

Thermoplastic materials are those that retain a plastic
nature after molding;  i. e., they can be resoftened by subsequent
heating.

The other group of plastics is known as thermosetting or
heat hardening materials.  These materials cure or vulcanize similar
to the curing of natural rubber during their molding operation.  In
molding, these materials undergo a permanent change which imparts a
heat resistance to them which does not permit them to be substan-
tially softened by additional reheating below their decomposition
points.

Due to the high degree of flexibility of the thermoplas-
tics, they are unsuitable for structural members where an apprecia-
ble magnitude of loading is involved.  From this standpoint alone,
they can be quickly eliminated from further consideration in the
molded form as load-bearing members;  however, they deserve consid-
eration in filament form as fabrics and rope to be used as tensile
elements in earth-supporting roof spans.

The thermosetting resins have several advantages which
suggest them for consideration as load-bearing members.  Basically,
they are in themselves not structurally strong materials and gen-
erally have a brittle nature.  Their greatest advantage in struc-
tural applications is that they can be combined readily with rein-
forcing fibers which impart to them the rigidity and high degree of
strength which they basically lack.  Thermosetting plastics which
are applicable are available either in an uncured liquid form or,
if solid, have a solubility in organic solvents.  This ability to
obtain these materials in a liquid state facilitiates the ease of

combining them with reinforcing materials such as fibrous strands or fabrics. When properly reinforced and cured, these thermosetting materials exhibit maximum stiffness for plastic materials.

As mentioned above, for simplicity plastic materials will be considered under two headings: Those suitable as structural load-bearing members; and those suitable for nonload-bearing usages.

a. Physical Properties of Reinforced Plastics and Those of Conventional Building Materials. In order to provide sufficient attenuation of gamma rays, it has previously been mentioned that the shelter will have to be covered with from 3 to 5 feet of earth. For the calculations, a earth weight of 100 pounds per cubic foot has been assumed. With a 5-foot overburden, there would be 500 pounds per square foot static pressure on the top of the structure. This would yield a pressure of $3\frac{1}{2}$ pounds per square inch on the top of the shelter.

In preliminary considerations, the ability of the structure to support this earth loading will be the primary concern; a secondary concern will be the problem of blast overpressure.

b. Advantages of Reinforced Plastics. Reinforced thermosetting plastics have some outstanding advantages as structural materials for shelter uses in comparison with wood, steel, aluminum and reinforced concrete but they also have, like all materials, some definite disadvantages. Their advantages for such usage are as follows:

(1) High Strength-Weight Ratio. Reinforced plastics can be made extremely strong in resistance to specific types of stress. Figure 5 shows a strength-to-weight comparison of specific tensile stress comparing the strongest form of glass-reinforced thermosetting plastic with conventional building materials and nonreinforced thermoplastics. To obtain the values, the ultimate tensile strength for each material was divided by density of the material. It will be noted that certain forms of glass-reinforced plastics have the highest strength-to-weight ratios obtainable today.

(2) Resistance to Ground Water. While glass-reinforced plastics are not unaffected by water for underground application, with proper choice of finish on the glass and plastic binder, the degree of strength deterioration can be limited to approximately 20 percent over a period of several years. Normal soil acidity or alkalinity has no appreciable effect on reinforced plastics.

| STRENGTH-TO-WEIGHT COMPARISON, PLASTICS AND CONVENTIONAL BUILDING MATERIALS | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Specific Tensile Strength (psi x $10^3$) | | | | | | | | | | | | |
| Material | 0 | 10 | 20 | 30 | 40 | 50 | 60 | 70 | 80 | 90 | 100 | 110 | 120 |
| Low Carbon steel | ▮ | | | | | | | | | | | | |
| Aluminum | ▮ | | | | | | | | | | | | |
| Nonreinforced thermoplastics | ▮ | | | | | | | | | | | | |
| Glass filament wound epoxy | | | ▬▬▬▬▬▬▬▬▬▬▬▬▬ | | | | | | | | | | |
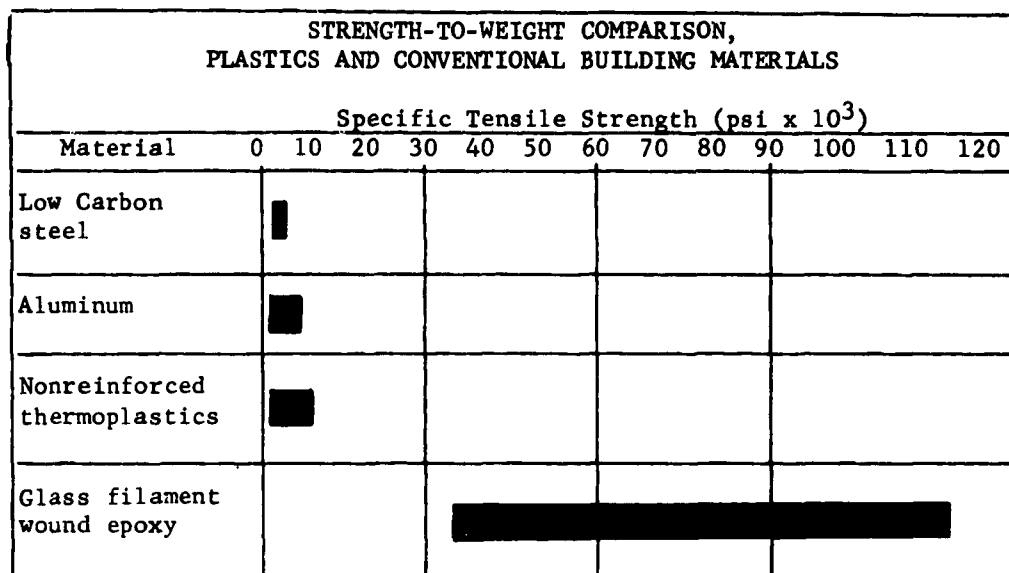
Fig. 5. Strength-to-weight comparison of plastics vs conventional building materials.

(3) <u>Ease of Forming into Complex Shapes</u>. This will be made increasingly apparent as the consideration of structures made from these materials is further developed. In this ease of formability, reinforced plastics have a significant advantage over steel, aluminum, concrete, or wood for the shapes of structures most advantageous for shelter use.

(4) <u>Ease of Handling</u>. Due to their lightness, reinforced plastic structures can be more easily handled without cranes or other special equipment than can those made from timber, steel, or aluminum. Coupled with this is the resistance to rough handling, abrasion, chipping, and fracture during shipping, outside storage, erection, and use. The use of reinforced plastics would, for example, permit the domed roof of a structure housing 20 people to be made in two pieces, each one of which could easily be carried by six men.

(5) <u>Low Thermal Conductivity</u>. An unheated underground structure made entirely from concrete, steel, or aluminum would maintain a uniform inside surface temperature of from 48 to 55° F throughout the year. Such a temperature is not a comfortable one for constant living conditions and would not be conducive to good health. The inside of a structure at this temperature would act as a condensation surface for moisture.

Plastics, on the other hand, have sufficient heat-insulating properties so that a structure made thick enough to support the earth overload would give sufficient insulation to prevent surface condensation and permit the body heat of the occupants to warm the air inside to a comfortable living condition during the winter.

    c.   Disadvantages of Reinforced Plastics.

        (1) Low Stiffness. In comparison with steel, aluminum, concrete, or wood, the stiffness or modulus of elasticity of plastics is quite low. Modulus is a ratio of stress in a material to its degree of bending or deflection. Structural steel has a modulus of approximately 30 million while the most rigid form of reinforced plastics has a comparable modulus of about 5. This means that when used in a span as a load-bearing member, the plastic will deflect at least 6 times as much as will steel under the same conditions. Reinforced plastics are not economical to use to support large loads over relatively long spans. The greater the load and the longer the span, the less economical they become.

        (2) Cost. Reinforced plastics cost considerably more on either a pound or an equal volume basis than conventional building materials.

        (3) Flammability. Although reinforced plastics can be made self-extinguishing, it should be pointed out that unless they are, there is always the danger of fire destroying them where they are used as load-bearing members.

        (4) Limited History in Structural Applications. Although reinforced plastics are expanding rapidly in their application in the building trade, it should be noted that almost all of these are, relatively recent and, therefore, comparatively little long-time durability data has been obtained from them. Due to the absence of complete long-time data which exists for conventional building materials, the designer of reinforced plastic structures has less to draw on.

    d.   Summary. In comparison with reinforced plastics, steel is extremely rigid and strong but has the disadvantages of being very heavy, is difficult to fabricate into complex shapes, is attacked by water, conducts heat readily, and is cumbersome to handle in large unitary structures.

Aluminum is lighter, has comparatively high tensile strength, and has about one-third of the stiffness of steel. Aluminum is also subject to corrosion by ground water and is difficult to fabricate into large structures with double curvatures.

Concrete has the advantage of being readily available almost anywhere in the country, is relatively inexpensive on a weight basis but is extremely heavy, and requires expensive forms for casting. Although it has good compressive strength, it is very low on tensile strength and requires steel reinforcement on the tension side of the load-bearing members.

Like the other conventional building materials mentioned above, wood is relatively inexpensive and is generally available throughout the country. It has the advantage of being light-weight but the distinct disadvantage of being difficult to fabricate into watertight structures of complex design. In addition, most grades of wood quickly absorb moisture and are warped or otherwise deteriorated by it when used underground unless given special pre-treatment. Under moisture conditions, wood is subject to rot, fungus growth, and termite attack. Its strength is fairly high on a weight basis, but wooden members are not ordinarily joined without considerable loss of strength in the joint.

Each of the above materials has very definite advantages and disadvantages for any building application. Reinforced plastics are certainly not without shortcomings, but they have sufficient inherent advantages to deserve serious consideration for underground shelter usage.

In the selection of any material, the question that the designer should ask is, "What material can do the best possible job for its cost, ease of fabrication, and installation?" This study has been conducted with a particular emphasis on practical considerations, and in this report the use of reinforced plastics will be compared on a price basis with other materials. The use of reinforced plastics cannot be justified unless they will do a better job at lower cost.

2. __Properties Desired in Shelter Construction Materials__.
Some of the properties desired in load-bearing members and other properties desired in shelter construction materials are as follows:

a. Ability of load-bearing members to support overburden.

b. Ability of these members to resist the effects of dynamic blast pressure on the surface.

      c.    Resistance of the structure to penetration during backfilling of earth or during the blast pressure wave.

      d.    Resistance to ground water penetration.

      e.    Imperviousness to rusting, corrosion, or other attack induced by moisture or high humidity.

      f.    Resistance to fungus, bacteria attack, or mildew.

      g.    Freedom from damage by termites and other insects.

      h.    Resistance to deterioration by moisture absorption reflected by swelling, warping, or delamination.

      i.    Freedom from attack from alkalinity or acidity present in the ground water.

      j.    Resistance to rapid heat loss through ceiling, walls, or floor.

      k.    Nonflammability.

      l.    Resistance to penetration of the structure by rodents.

With the above requirements in mind, the materials that are available for use in reinforced plastic combinations should be considered.

      3.    Choice of Reinforced Thermosetting Plastics for Structural Members. The fibers used for reinforcement will first be considered. It has been mentioned earlier that without fibrous reinforcement the thermosetting plastic materials are very poor in strength and generally lack the required rigidity. They are brittle and lack tensile, flexural, and buckling strength. The fibrous reinforcement in these combinations with plastics act much in the same way as steel reinforcing rods in concrete. Once the reinforcement is added, the combination then assumes much of the strength of the fibrous reinforcement with the plastic largely acting as a stable binder to physically hold the reinforcement in position. Without such reinforcement, the thermosetting plastics have no properties to suggest them for serious consideration as load-bearing members. With the reinforcement added, they can then become the strongest materials known today on a strength-weight basis.

      The types of fibrous reinforcement that have been used for this purpose are as follows:

Glass.
Cotton.
Asbestos.
Nylon.
Regenerated cellulose, rayon, and Fortisan.
Acrylic fibers such as Orlon and Dynel.
Polyester fibers (Dacron).
Comparatively large organic fibers such as jute,
    hemp, and sisal.

The two principal properties most important to the fibrous reinforcement are high tensile strength and low modulus. It is also important that the fibers be available with comparatively uniform diameter and either in continuous filaments or a long staple.

Glass filaments have so little competition in meeting the above requirements that more than 90 percent of all reinforced plastics used today employ glass. The other fibers listed above are used to reinforce plastics only where the structural strength and stiffness of the combination is not a primary requirement.

Having chosen glass as our reinforcing material, it is appropriate at this time to examine the various forms in which it is available for the reinforcement of thermosetting plastics. Table XIV lists these forms.

## Table XIV

| Glass Form | Max. Glass Content Used in Plastic Reinforcement (% by weight) | Price Range (per lb) |
|---|---|---|
| Continuous Roving | 90 | $ .30 - $ .60 |
| Yarn | 90 | .40 - .65 |
| Cont. swirl strand mat | 50 | .50 - .67 |
| Spun roving | 50 | .32 |
| Fabric (181 Series) | 75 | 1.20 - 1.80 |
| Fabric (unidirectional) | 75 | 1.20 - 1.75 |
| Fabric (1000 Series) | 65 | 1.00 - 1.25 |
| Woven roving | 60 | .65 - 1.00 |
| Woven spun roving | 55 | .55 - .75 |
| Chopped strand mat | 50 | .48 - .65 |
| Mechanically needled mat | 50 | .60 - .65 |
| Chopped spun roving | 50 | .32 |
| Surfacing and overlay mat | 20 | 1.70 - 1.80 |
| Milled fibers | 15 | .40 - .45 |

The overriding factors influencing the choice of the form of glass are the strength that it imparts to the combination, its adaptability to low cost manufacturing operation, and its cost. The primary consideration will, therefore, be with glass roving and the processes that use it.

Equally important to choosing the proper form of glass is a choice of surface treatment on the filaments.

a. <u>Surface Treatment of Glass Filaments</u>. Before the proper type of surface treatment to obtain maximum adhesion to the plastic employed is considered, it is well to have a brief knowledge of the glass filament manufacturing process. Glass fibers are usually made by mechanically drawing a filament from a stream of hot, molten glass. Immediately after forming and cooling, a chemical treatment called a "size" is applied to the surface of the glass fiber.

The size has two functions: to protect the fragile filaments from abrading and breaking each other and to give a chemical coupling to the plastic to be used in the laminate. After the size has been applied, these filaments are then collected into a bundle known as a "strand" in a gathering device.

Beyond the gathering device, the strand is wound into a forming package. The forming package is a fragile container of wound strands from which more sturdy forms of fibrous glass are produced in subsequent operations. Roving consists of a definite number of strands that are brought together and wound on a spool in a continuous parallel arrangement.

Since glass filaments are too fragile to be handled without surface treatments, these have an important effect on the strength of the final molded item. First, they permit greater strength by minimizing the breakage of filaments in handling and molding and, secondly, some surface treatments exhibit a marked chemical affinity to specific plastic resins. These treatments are designed to accomplish a chemical bond between the glass and the plastic which contributes to maximum strength and water resistance in the final molding. Other surface treatments show little or no affinity to specific plastics and yield both poor strength and moisture resistance in combination with these plastics.

The differences in the performance of laminates due to differences in size or finish employed can be sizable.

At present, alkyloxysilane coupling agents are becoming available as components of glass roving sizes (or binders) and are preferred for this application due to the long-term water resistance that they impart to the glass-to-polyester resin bond. The initial dry strength resulting from use of these agents may be somewhat lower than that resulting from use of other coupling agents, but the strength after long exposure to water will be measurably greater.

After glass filaments have been selected as reinforcement and consideration has been given to the choice of a suitable surface treatment for them, the choice of an appropriate resin must be made.

b. <u>Selection of Plastic Resin</u>. The function of the plastic in a fibrous combination is to provide physical support and spacing to the reinforcing filaments and hold them in place while the combination is being stressed. In high strength glass resin combinations, the resin contributes considerably less strength than the glass. The thermosetting resins used most commonly with glass reinforcement are as follows:

(1) Polyesters.

(2) Epoxies.

(3) Phenolics.

Although the phenolic resins are the least expensive of the three, very little consideration will be given to them for this application due to the fact that they are normally solids and must be dissolved in a solvent such as alcohol to liquify them. It is in a liquid form that thermosetting resins are normally combined with glass reinforcement.

If a closely woven glass fabric were employed for this purpose, it could be saturated with phenolic resins from solution and then dried to remove the solvent. However, no such convenience exists in the use of more open reinforcements such as chopped roving.

Phenolics have another important disadvantage for this purpose. In curing, they go through a chemical reaction known as condensation which produces moisture. To accomplish proper curing of a phenolic laminate, it is generally necessary to use two-part molds and use pressure in addition to the heat required for curing. This pressure is necessary to accomplish the curing reaction and to remove moisture in the form of steam.

In contrast, however, both the epoxies and the polyesters are liquid resins which cure simply by addition of one molecule to the other without the formation of by-products such as water. Their initial viscosities are generally ideal for rapid wetting of the glass strands in the molding process.

Although epoxy resins are known to give the highest strengths with glass reinforcement, where this is not the overriding consideration polyesters are often employed because of their considerably lower price. At present, polyesters are selling for as low as 28.5 cents per pound while epoxies are approximately 60 cents per pound. Since the difference in strength is comparatively small and the difference in cost is large, polyester resins are used in about 85 percent of all reinforced plastics. Epoxies are selected for applications where the maximum strength per pound is required and where cost is a secondary consideration.

For this reason, we are selecting polyester resins for this particular application. Since these resins are available in different types ranging from extremely flexible to very rigid, it is important to this application to select a rigid resin in order to achieve the greatest strength and highest modulus.

4. <u>Manufacturing Processes</u>. As a reinforced plastics shelter might be regarded from a process standpoint as a large inverted boat in the shape it approximates, it would appear logical to first give consideration to processes that either have or are now being used in the manufacture of large boat hulls.

One of the earliest processes used for making boats employed a female mold made from wood and covered with metal or plastic. In this form, the hull was laid up usually beginning with a layer of fabric on the outside followed by layers of chopped glass strand mat. As each layer was put on, it was saturated by hand with a polyester resin and rolled using a small metal roller to remove air bubbles. After the proper thickness had been built up over the entire surface, the laminate was then permitted to cure.

The polyester resins used in this process were generally catalyzed for room temperature curing. Curing was done using two different methods. Employing the process known as contact molding the laminate is simply allowed to cure after the rolling operation. The second process is known as vacuum molding and subjects the entire laminate to a vacuum by a means of a thin "bag" of flexible plastic film that was sealed around the periphery of the mold. A small vacuum pump is connected to the bag. The effect of this vacuum was to remove additional air and to employ the normal atmospheric pressure to assist compaction of the laminate during cure.

The next method developed employed a large perforated metal male collecting screen in the shape of the hull of the boat. The purpose of this screen was to make a preform using chopped glass roving which was less expensive than mat.

In the process, a strong vacuum was applied to the perforated form while an operator sprayed the form using a hand "gun" which chopped roving continuously supplied to it into approximately 2-inch-long strands. These strands were sucked into the screen and collected there in the proper thickness for the final hull.

After all of the glass had been deposited and while the suction was still held on the screen, the preform was then sprayed by hand with a water-dispersed binder. When this binder had been thoroughly dried using hot air, the preform then had sufficient structural strength so that it could be removed from the screen in one piece and laid into the female mold. It was then saturated by hand with a polyester resin and molded in the previous manner.

The next step taken by one boat manufacturer was to go to the rather sizable expense of having a completed matched metal mold machine so that the preform described above could be molded under relatively high pressure. This process has the advantage of producing a very dense void-free molding with two finished surfaces, with a minimum of labor. The obvious disadvantage of the process is the extremely high tooling cost. Matched steel molds could only be considered if the annual production volume was extremely high and the runs long.

The most advanced process for boat manufacture developed to date employs a "gun" which supplies chopped roving and sprayed catalyzed polyester resin to the boat mold simultaneously. Three materials are furnished to the gun in this process. The first is continuous glass roving, the second is a polyester resin, and the third is a catalyst or curing agent that accomplishes room temperature curing of the plastic after the deposition on the mold has been completed.

In most operations to date, the gun is handled manually by an operator who must judge by eye the proper thickness of material being deposited. However, some boat manufacturers are beginning to mechanize the operation so that the mold moves continuously down a conveyor while fixed "guns" spray glass and polyester resin into them. After spraying, the build-up must be rolled to remove voids and compact the glass-resin combination.

The justification for mechanization of this operation is strictly one of volume of manufacture. For the volume obviously anticipated for community shelters, this approach should receive serious consideration.

It is felt that the gun method of deposition has the most promise for shelter manufacture. The manner in which it is used will depend on shelter design and production volume.

It would be preferable to spray onto a male form or mold, producing a finished surface on the inside. A colored gel coat of polyester resin would first be sprayed onto the mold to yield a water seal and give an attractive appearance to the inside shelter surface. This would be followed by the glass and resin spray which is applied in layers and then rolled.

In the spray-up process just described, glass roving is cut into short lengths by the gun for convenience of incorporating into the resin-bonded structure. However, for many designs, it is also quite feasible to consider a method of manufacture in which the roving is used in a continuous form without chopping.

a. Roving Winding. The original form of this process is known as filament winding and employs glass strands which are unwound from a spool or forming package and go through a resin bath where either an epoxy or a polyester resin is added before the glass strand reaches the winding form. Common shapes of winding forms are spheres and cylinders with convex ends. The winding form is rotated in the process so that with a sphere, a baseball type of wind would be produced. With a cylinder, the form would also be rotated but the winding mechanism would be oscillated to produce a helical wind on the form.

This process is significant in that it uses glass in its least expensive form and in such a way that the combination of glass and resin produced has the highest strength-weight ratio of any materials known today. The normal glass content in filament wound parts ranges from 70 to 90 percent. Another advantage of the process is that it lends itself readily to automation.

From the standpoint of cost per pound, it would probably be desirable to employ a polyester resin for a shelter made by this process. The resin could be compounded so that the part would cure at room temperature on the form after it had been removed from the winding station. The form would necessarily have to be designed in parts so that it could be disassembled and removed in separate pieces from an open end of the cured filament wound structure.

With regard to specific designs adaptable to shelter use, a helically wound part made in the shape shown in the accompanying illustrations would be practical to consider for this process.

Figure 6 shows a shape comprising an arched roof, straight sides, and a flat floor that could readily be wound in short sections and assembled, end-to-end using internal flanges. These flanges would have the obvious advantage that they could be further tightened from the inside if ground water leaked into the interior.

These sections would be made in 4-foot lengths with a width of approximately 12 feet and a maximum ceiling height of 8 feet. With a 3/8-inch wall thickness, they would weigh approximately 500 pounds each.
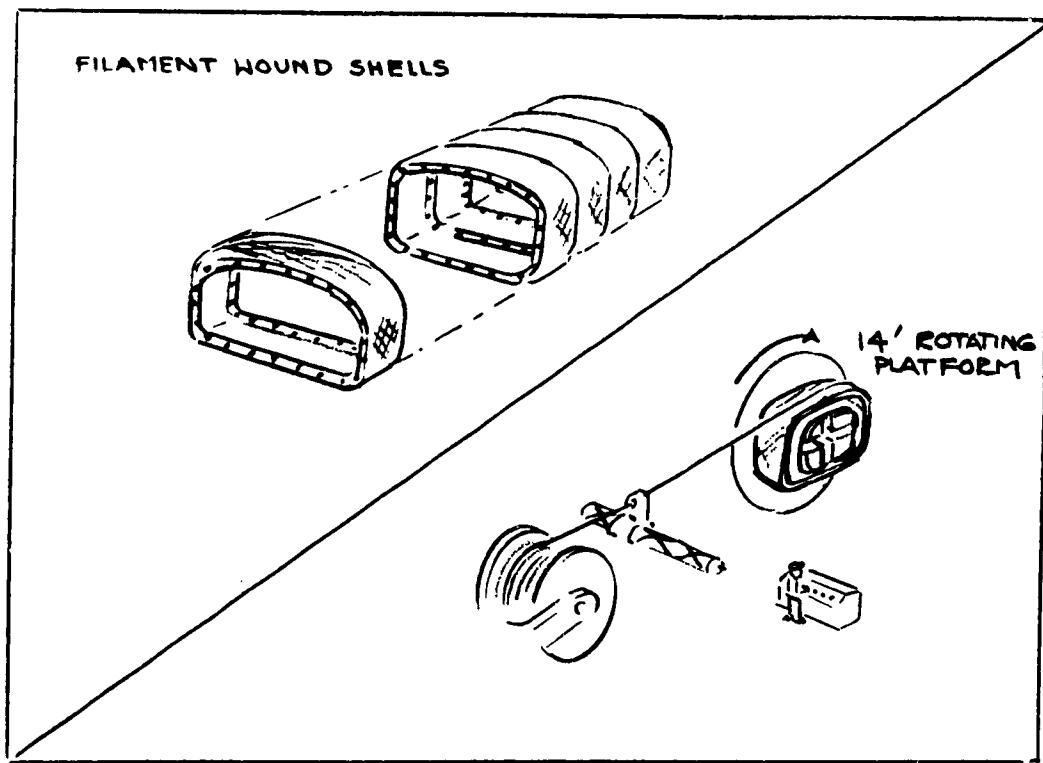


Fig. 6. Section shapes - design 6.

Figure 7 shows a profile section of one of the modular units of this shelter. The 8-foot center ceiling height would provide ample head room for standing except in the area near the

126

side walls. A notable disadvantage of this modular shelter is that
bunks placed lengthwise along the wall would have to span one or
more of the joining flanges. This limitation would present maximum
utilization of internal space.
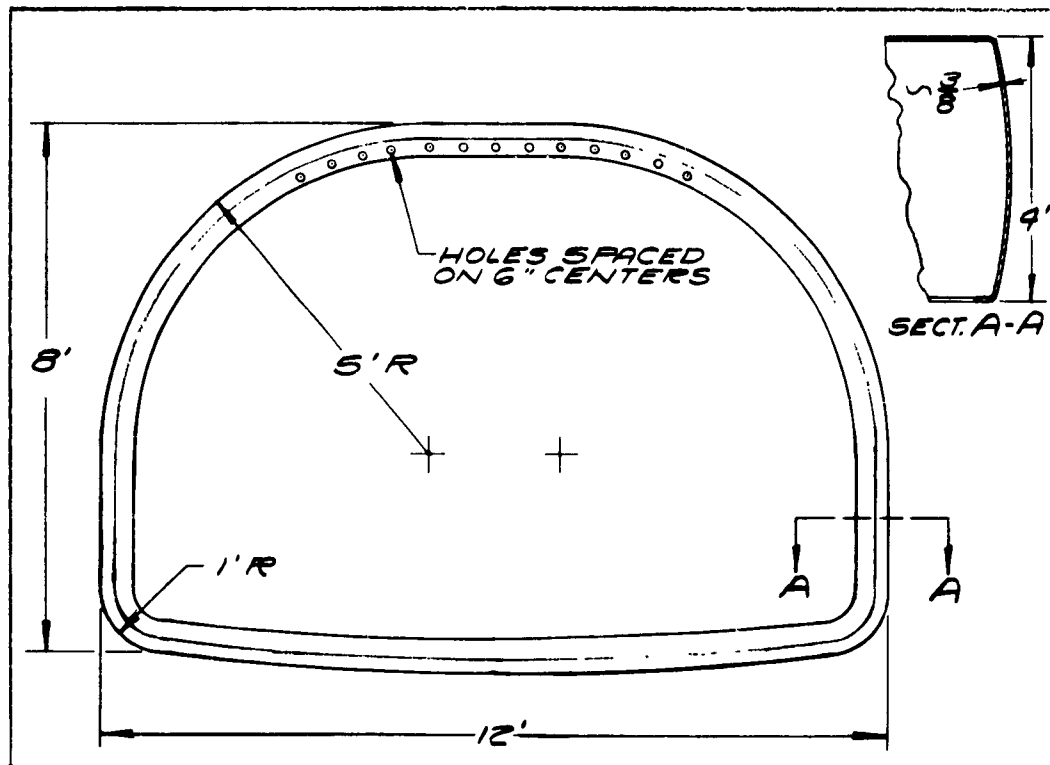


Fig. 7. Profile section of modular unit - design 6.

The full round section (Fig. 8) would be 10 feet in
diameter and 6 feet long and would weigh approximately 600 pounds.
The cross section shows the placement of storage sections and bunks
capable of sleeping six with space available for passage and
standing.

At present, the cost of filament windings using
epoxy resins is approximately $1.00 per pound. However, using com-
paratively heavy rovings, special compounding of polyester resins
and highly automated techniques employing improvements in this proc-
ess, it might be possible to compete on a pound basis with the gun
roving laminate. This could be accomplished only after the required
development effort had been completed and the operation tooled for
extremely high production volume.
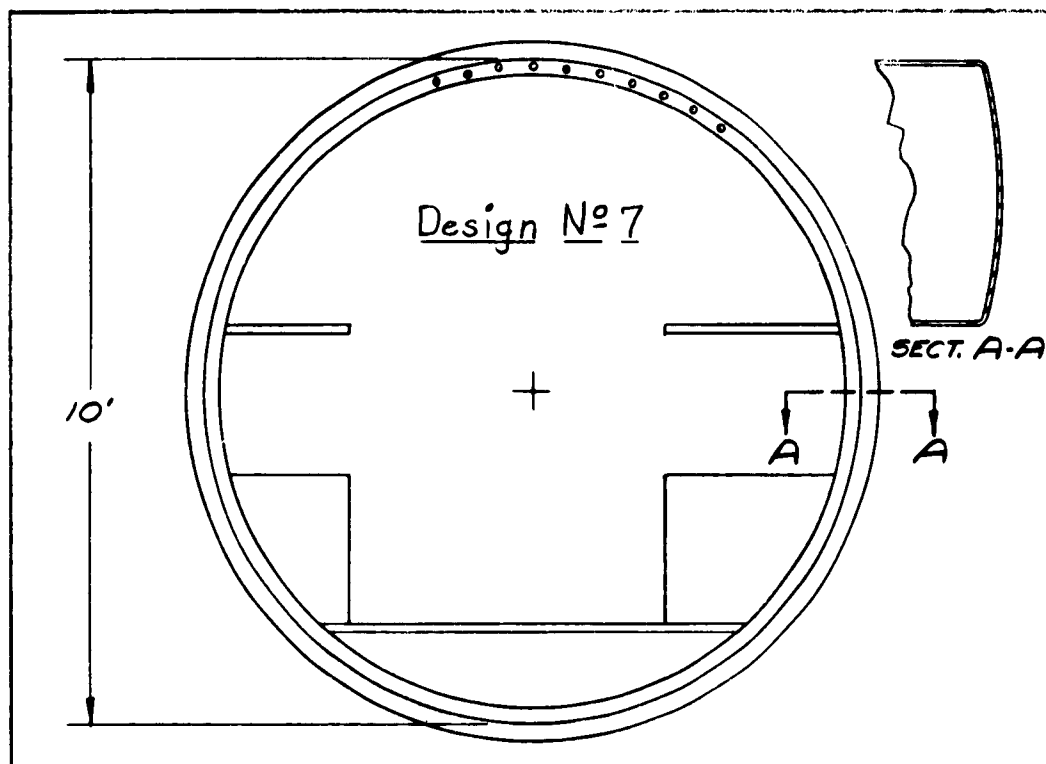
Fig. 8. Full round section - design 7.

With regard to the tooling required, it should be noted that in contrast to the gun spray-up process, comparatively few firms in this country are set up to make an item as large as a fallout shelter employing filament windings. To employ the roving winding technique in production would require special setups and considerably more expensive tooling than is required for the spray-up process.

b. Casting of Inexpensive, Low-Density Structural Compounds. An attractive possibility for shelter construction is the concept of casting a lightweight mixture into rather massive arched shapes which are quite narrow and lightweight and which could be assembled side-by-side with minimum effort. A possible design of section is shown in Fig. 9.

Normally, concrete would be used for this type of construction, but the excessive weight of concrete would require rather involved equipment for the erection of any practical width and thickness of arch. An arch of the size shown in Fig. 9 would weigh 6,600 pounds if cast from conventional concrete.
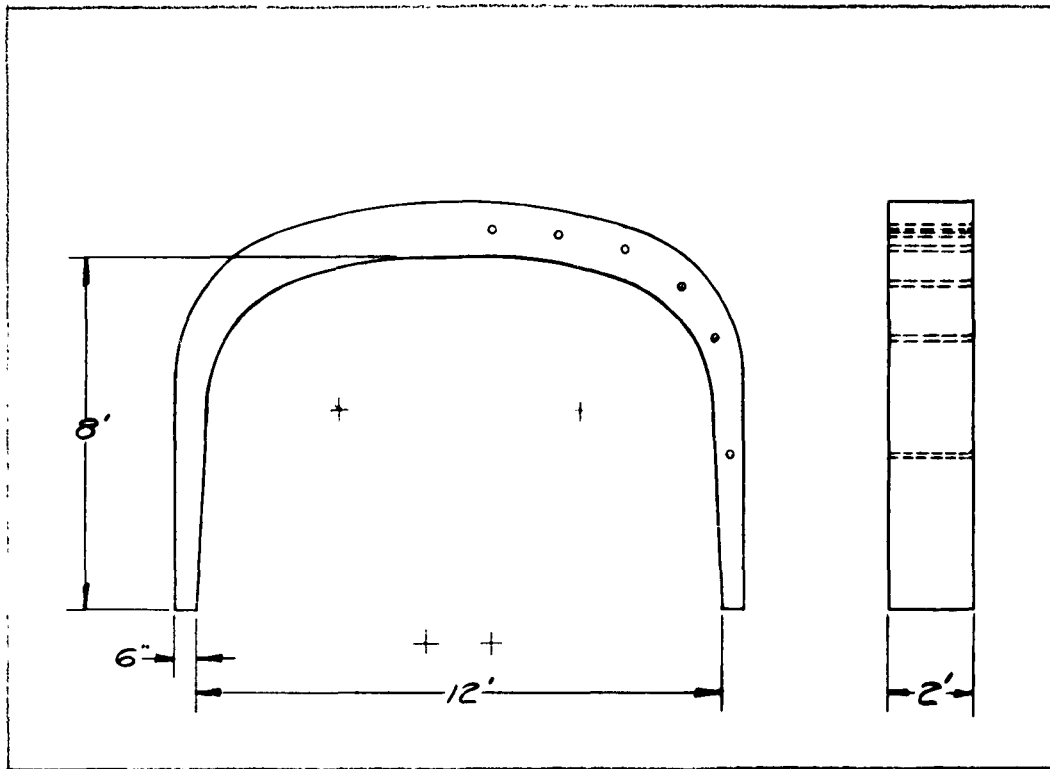
Fig. 9. Solid cast arch - design 9.

If a suitable mixture could be developed, it might be cast into simple wooden forms that could, after handling, be erected by hand and assembled into a continuous arched roof shelter as shown in Fig. 10. The profile area of the arch is about 23 square feet so that a 1-foot-thick casting would have the volume of 46 cubic feet.

The casting mixtures which are being considered should have a specific gravity of less than 0.5. These mixtures would be the low-cost combinations of binder or cement and filler material. Some binders and cements which were used in early work of this type are polyester resins, polyvinyl acetate, Portland cement, water glass, and other inexpensive resins and glues.

The fillers used could be plastic microballoons, foam beads, wood chips, sawdust, or chopped fibers.

Compounds of the type described which would be suitable for this application are not available today but might be developed. This possibility is covered further in the section of this appendix dealing with proposals for further development effort.

OPTIONAL
2 PART ARCH

VINYL COVER

SEALER

STAGGERED
BOLTED

CONCRETE BASE

TIMBER OR
CONCRETE ENDS

CAST IN GROUND
OR WOOD FORM
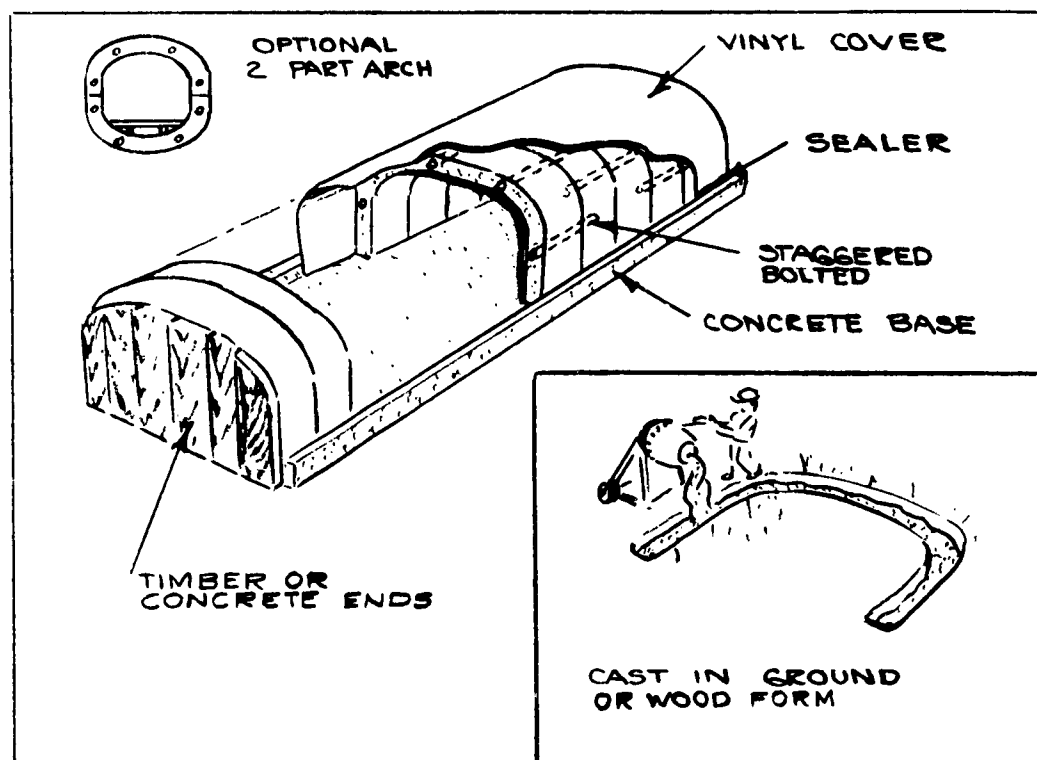
Fig. 10.  Continuous arched roof - design 9.

c.  Consideration of Approaches to Shelter Manufacture from a Location Standpoint.  In order to provide background for this discussion, it will be desirable to briefly describe two processes that have recently been developed.

The first of these processes was developed by the Air Force.  This process uses the spray gun described above for fabricating housing structures outdoors in the field using a lightweight mold that is fabricated from thin plastic film.  This mold is inflated by means of a small blower and is used as the male form onto which the glass and resin are sprayed.  This spray-up is then rolled by hand and allowed to cure.

The advantages of this process are:  (1) Minimum transportation cost;  (2) extremely low cost and conveniently transportable tooling;  and (3) a minimum of readily transportable equipment required.

The disadvantages of the process are: (a) Outdoor hazards such as wind, rain, and changing temperatures will probably make this process impractical to use during most of the year; (b) the required equipment is not now available mounted on trucks throughout the country; (c) an operator skilled in the spraying operation would have to accompany the truck; and (d) an inflated form would have two inherent disadvantages (i. e., the limited inflation pressures would hamper the effectiveness of the hand rolling operation after spraying, and the inflated bag would necessarily restrict the design possibilities).

The second process is the "Buildings in Barrels" concept developed by USAERDL. The raw materials are glass roving, polyester, and urethane resins. The process employs a series of lightweight reinforced plastic molds which are used in a heated building near the installation site.

Using these two-part molds and moderate clamping pressures, a series of sandwich components are produced for assembly. The process uses the spray guns described above for spraying chopped glass roving and polyester resin skins together with the secondary operation of spraying a rigid urethane foam as a sandwich interlayer. The strong, lightweight building components are then readily field assembled using adhesive sealers.

Both of the above processes have been devised to simplify the problem of erecting personnel surface shelters in the field. These processes further demonstrate the extreme versatility of reinforced plastics. The first presents the possibility of fabricating the shelter at the location where it will be installed, using simple portable equipment. The second process suggests the possibility of employing the same equipment together with portable rigid molds. This equipment could be operated in a distribution area until it had produced all of the shelters required for that area after which it could be moved to another.

For this purpose, it would probably be employed to produce rather readily transportable components which could be assembled at the installation site.

Thus, there are three possibilities: manufacturing in plants now in existence, manufacturing at a distribution site using a portable plant, or manufacturing in large units outdoors at the shelter installation location.

It is felt that the manufacture of reinforced plastic shelters requiring the spraying and curing of the structure outdoors

is not practical during most of the year for a majority of locations in this country. Such a process should be used only in an enclosure which would exclude wind, rain, and snow and provide either complete heating or partial heating of the temporary enclosure to achieve some stabilization of temperature.

The use of a-portable plant is a definite possibility, but its principal advantage of limiting shipping costs is largely overcome by the fact that there are approximately 165 manufacturing plants in this country that presently make reinforced plastic boats (and could just as easily make shelters or shelter components); these plants are distributed through most of the States (see Appendix H). Most of these are equipped with glass and resin spray guns. Those that are not so equipped can obtain them at nominal cost.

In mid-1961, there were approximately 750 spray-up depositor guns in use in this country. These guns are capable of applying about 3 million pounds of glass roving and about 7 million pounds of polyester resin per year. Thus, in mid-1961, there was available in this country in manufacturing plants equipment for spraying 10 million pounds per year of glass-reinforced plastic of the type being considered for shelter manufacture. This capacity has undoubtedly been significantly increased in the interim by manufacture of additional guns.

Up to this point, the primary concern has been the application of glass-reinforced thermosetting materials in load-bearing structures. These are by no means the only plastic materials that should receive consideration in a community fallout shelter.

5. **Plastics and Rubber for Nonload-Bearing Uses**. Thermo-plastics will first be considered for such applications.

a. **Vinyls**. The vinyls constitute a family of materials ranging in properties from relatively hard and rigid compounds to soft, resilient, flexible rubber-like elastomers. The rigid vinyls might be considered as an interior lining for walls or ceiling.

A medium flexibility of vinyl could be employed as a flooring material if it were installed on a relatively rigid basis. The very resilient flexible vinyls might have application as extruded ceiling gaskets between joined component parts of the structure.

Vinyl-coated aluminum has proved itself in the building field as house siding. A vinyl-coated steel sheet has been developed which will stand the elongation required in the forming of the steel up to 30 percent without failure. Thus, even if the body

of the shelter were to be made from aluminum or steel, coatings or polyvinyl chloride resins would materially contribute to their practicality.

b. <u>Polyolefins</u>. Polyethylene and polypropylene are sister plastics and possess superior toughness, flexibility, and chemical resistance. Their low cost, low moisture permeability rate, and availability in thin films suggests their use in large sheets around the entire exterior of the shelter as a moisture barrier. Polyethylene has already been proved in the building industry for such applications.

c. <u>Asphalt</u>. Another well-known naturally occurring thermoplastic, asphalt, may be worthy of consideration either as a flooring material or in mastic cements to seal joints in assembled structures.

d. <u>Polystyrene</u>. Polystyrene has the lowest cost of the thermoplastics and should be considered in its expanded form as a core material in sandwich constructions. Expanded polystyrene is worthy of consideration for such a purpose due not only to its relatively low cost but also to its insulating properties and low moisture absorption. It can also be made into large panels of most any shape at low cost.

e. <u>Rubber</u>. Three types of rubber would be suitable as sealing gaskets. These are natural, GRS (Buna-S), and Neoprene. Natural rubber and GRS would be less expensive for this application, but Neoprene has an advantage worthy of note. In contact with moisture, it will absorb 50 percent of its own weight without significant deterioration of properties. This would mean that if it were used as a gasket and a proper seal was not made by tightening the joint sufficiently during installation, subsequent moisture absorption would swell the gasket an additional 50 percent. This additional swelling would very likely accomplish the desired seal.

f. <u>Epoxies</u>. To return again to the thermosetting materials, the versatility of epoxy resins may make them applicable for more than one specific use in a shelter. The outstanding properties that make the epoxies so universally adaptable is their outstanding adhesion to almost any other material, coupled to their chemical inertness. Epoxy resins are presently being used as flooring compounds which can readily be trowelled in place. They are also employed as mortars for use with concrete blocks. In addition to these uses, they also have application as corrosion-resistant coatings for steel and aluminum

g.  <u>Polyurethanes</u>.  We have already mentioned the polyurethane materials as plastics that can be foamed into place as rigid materials applicable either as an exterior insulating layer or as a core for high-strength, light sandwich constructions.  This expanded plastic also has the advantages of resistance to moisture and superior adhesion to sandwich skins.  Equipment for foaming polyurethanes in place in hollow structures is readily available in most areas of this country.

6.  <u>Sandwich Structures</u>.  As mentioned previously, the structural sandwich panel consists of sheets of dense material bonded together with a comparatively low-density core between them.  Each of the components exhibits particular properties and contributes these as part of the over-all composite structural strength.

The comparatively thin outer layers or skins of dense material have high tensile and compressive strengths.  These skins must be able to accept high stresses induced by bending loads.  The most commonly used sandwich skins consist of fibrous glass reinforced with polyester or epoxy resin.  As in straight moldings, the polyesters are by far the most extensively employed.

Although more expensive and more difficult to handle than polyesters, epoxy resins make the strongest skins and give maximum adhesion to the core material.  The resulting sandwich is also stronger than those made with other resins.

Metals, such as aluminum and stainless steel, have been used as skin material but these are not felt to be particularly applicable in a shelter.

The core used in sandwich construction is a rigid material with high compressive strength and low density.  These properties are achieved by employing either expanded plastics or a honeycomb core made from kraft paper, aluminum, or even, in some instances, stainless steel.  Kraft paper is by far the most popular material for this usage.  It is broadly employed in aircraft construction.

Honeycomb is made by impregnating kraft paper with a rigid resin such as a phenolic and bonding strips together discontinuously in a regular pattern.  The cured material can be expanded by pulling the two sides apart with the resulting structure looking like the paper Christmas bells that open from a flat shape.  When bonded between facings of a structural sandwich using an adhesive, the honeycomb provides good lateral stability and prevents buckling under loads either perpendicular to the facings or in their plane.  Paper honeycombs are not felt to be particularly adaptable to underground

use because they are expensive to fabricate into curved shapes and are subject to considerable moisture absorption with corresponding decrease in strength. Although honeycomb is not particularly promising as a core material, the same is not true of expanded rigid plastics such as the polyurethanes and polystyrenes which do not have the same limitations.

The USAERDL buildings-in-barrels development previously mentioned employs molded sandwich panels with reinforced plastic skins over a core of polyurethane expanded plastic. The primary purpose of the foamed plastic in these buildings is for heat insulation. This sandwich construction could be used in an underground shelter by employing comparatively thicker surface layers of reinforced plastic and a high-density urethane foam. Either high-density expanded polystyrene or balsa wood could also be used as a core.

A disadvantage of the foam sandwich construction is that it would provide too much heat insulation. During the summer, while the shelter is crowded with occupants, the inside air temperature could rise to a dangerous level unless the shelter was thoroughly ventilated.

7. <u>Plastic Fibers and Their Fabrics</u>. A number of the thermoplastics are well known for their successful use in the filament form. The fibers employed in fabrics are of two types: discontinuous (staple) and continuous filaments.

These fabrics are woven, knitted, or felted. The most commonly used are nylon, Saran (polyvinylidene chloride), acrylonitrile polymers and copolymers (Orlon, Vinyon, and Acrilan), polyester (Dacron), polyethylene, and cellulose esters (acetate and Arnel). Regenerated cellulose or rayon is also sometimes considered as a plastic. A very high-tenacity form of this fiber is known as Fortisan.

Of these, Fortisan and nylon have the highest tensile strength. Although these fibers have a high tenacity, they are not recommended for the reinforcement of plastics structural components due to the fact that they have a relatively low modulus which permits them to stretch considerably in comparison with glass before they begin to assume a large proportion of the load. Such a characteristic in a laminate reinforcement contributes measurably to its deflection under load. Although these synthetic fibers do not appear promising from the standpoint of reinforcing a plastic, they deserve examination in woven fabrics as a roof to support the static earth overload. For this use, the fabric would be given a waterproof coating of vinyl plastic.

In this type of application, the shelter would be a trench provided with a floor and retaining wall made from conventional building materials. The roof would be a light, readily portable structure of coated fabric held rigidly over the wall of each side. The fabric supporting members could be made from a variety of different materials such as aluminum tubing, reinforced plastic beams, or heavy nylon webbing.

Woven fabrics would be chosen for this purpose rather than knitted or felted fabrics due to their superior tensile strength and limited extensibility. Possible designs of earth supporting roof shelters are considered under the Design section of this appendix.

After this consideration of plastic materials for shelter use in the various forms in which they are available, the form or design that the shelter will assume is discussed.

8. General Design Considerations.

a. Geometric Shapes. Numerous geometric shapes are available for the shelter design. In consideration of shapes, these configurations can be classified into geometric surfaces and geometric solids. The geometric surfaces can be sub-classified as shown in Fig. 11.

GEOMETRIC SURFACES

| Ruled Surfaces | | Double Curved Surfaces |
|---|---|---|
| Single Curved Surfaces | Warped Surfaces | |
| Cylinder Cone Convolute | Helicoid Hyperboloid of one Nappe Hyperbolic Paraboloid, Conoid Cylindroid | Sphere Torus Oblate Spheroid Prolate Ellipsoid Oblate Ellipsoid Paraboloid Hyperboloid of two Nappes |
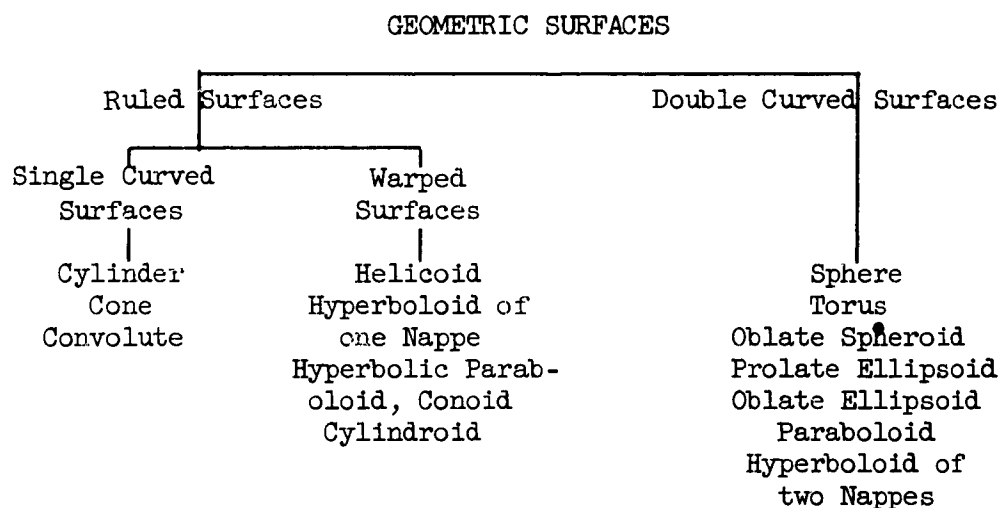
Fig. 11. Geometric surfaces sub-classified.

A single curved surface is one that can be unrolled to form a plane (i. e., it is one that can be developed). A warped surface, by contrast, cannot be unrolled to form a plane surface and

is not developable from a flat sheet. A double curved surface is one generated by a curved line. Figure 12 illustrates a number of these geometric forms.
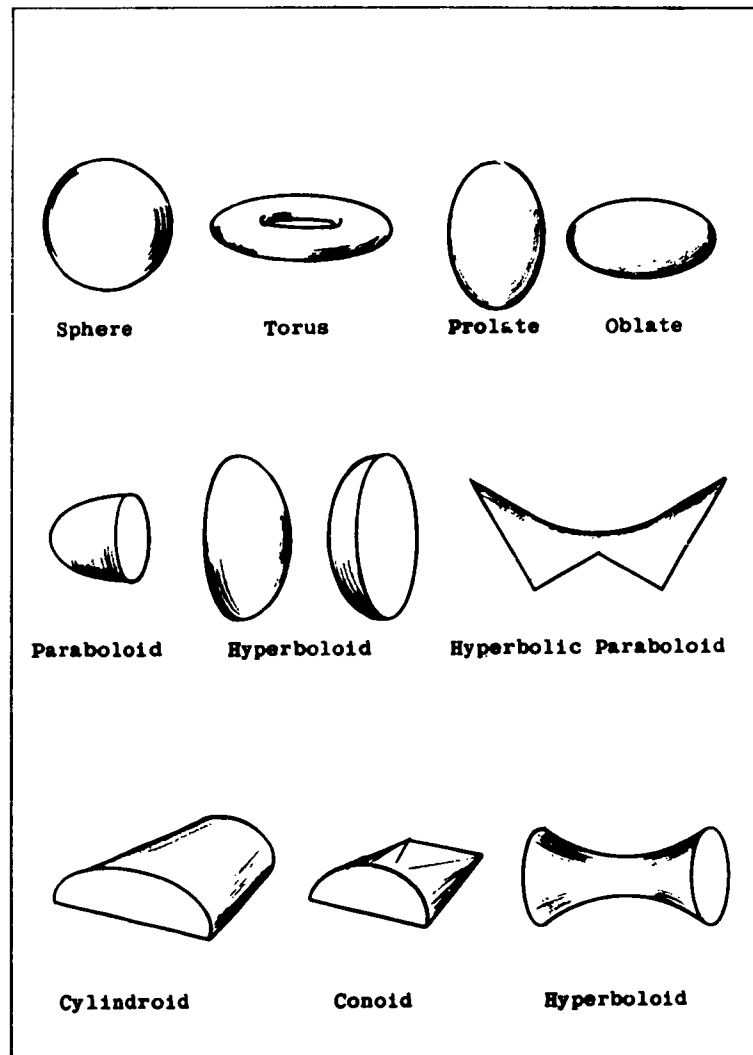


Fig. 12. Some geometric shapes.

Brief consideration is given below for each of the above geometric surfaces for possible shelter use.

(1) <u>Cylinder</u>. This shape is worthy of detailed consideration largely when halved along its axis and used as a roof member.

(2) <u>Cone</u>. Not advantageous either from a structural standpoint or convenience of useable internal volume.

(3) <u>Hyperboloid</u>. From the standpoint of usability of internal volume, this shape is not particularly desirable but it is possible that a sound structure could be made by utilizing portions of it as a roof. From an over-all standpoint, it is not felt to be particularly promising.

(4) <u>Conoid</u>. This shape has little to offer from the standpoint of useable internal volume.

(5) <u>Sphere</u>. Portions cut from a sphere are worthy of serious consideration. The hemisphere would make an ideal roof for a round shelter. One of the designs pictured further in this appendix is made up of joined sections of spheres.

(6) <u>Torus</u>. This would be a structurally strong shape, but internally it would not be particularly convenient.

(7) <u>Oblate Spheroid, Prolate and Oblate Ellipsoid</u>. Each of these are structurally sound shapes, and the internal volume is sufficiently useable. Of the three, the oblate spheroid is probably the most convenient for manufacture and is given further consideration for shelter use in this appendix.

The geometric solids can be classified as shown in Fig. 13.

GEOMETRIC SOLIDS

| Polyhedra | | Solids of Revolution | | Solids Bounded by Warped Surfaces |
|---|---|---|---|---|
| Regular Solids | Irregular Solids | Generated by a Straight Line | Generated by a Plane Curve | |
| Tetrahedron Cube Octahedron and Others | Prism Pyramid and Others | Cone Cylinder | Sphere Spheroid Torus Paraboloid Hyperboloid | Conoid Cylindroid Hyperboloid of One Nappe |
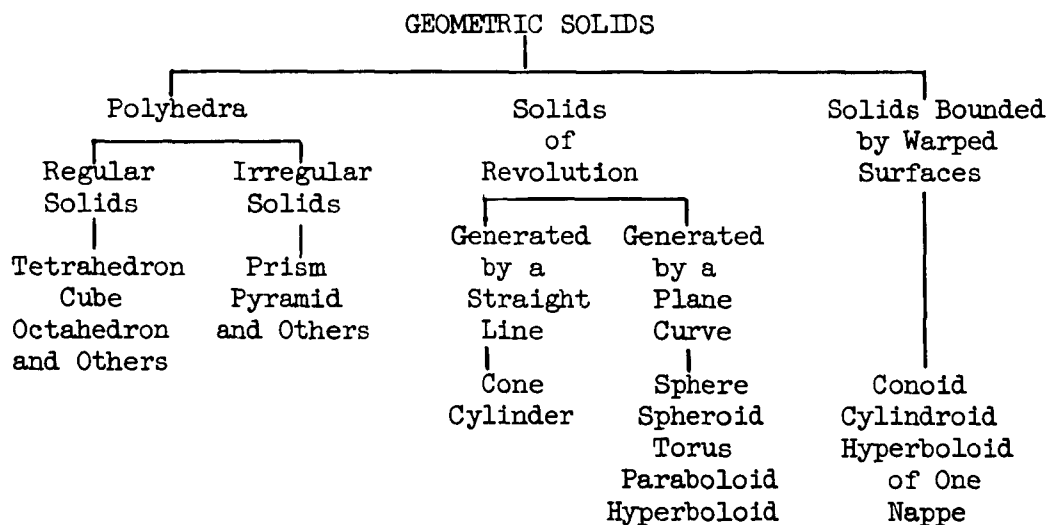
Fig. 13. Geometric solids classified.

138

It will be noted that there is some duplication between the tabulations of geometric surfaces and geometric solids. The following comments refer to those shapes which were not considered in the above discussion of geometric surfaces:

Polyhedra are bounded by plane or flat surfaces as bases and can be eliminated for this reason. Reinforced plastic members essentially lack stiffness in this form and as such are inefficient structurally. A more efficient configuration for reinforced plastics is the doubly curved surface, of which the sphere is an example.

Another would be the curved corrugated surface commonly seen in the quonset-type hut. Many other forms of doubly curved surfaces are being used in modern architecture.

These doubly curved surfaces make up for the comparatively limited stiffness of reinforced plastics by providing them with rigidity that is inherent in this shape. The effect of the double curvature is to decrease the thickness of reinforced plastic material that would be necessary in an arch section.

In consideration of basic structural shapes, they should be looked on from the standpoint of their inherent structural stability, the usability of the internal volume, and their convenience for manufacture, shipment, assembly, and installation. In addition to these important considerations, there is one other fundamental one that should be included.

b. Dependent and Independent Shapes. To understand this means of classification, basically fundamental to buried structures, it is important to note that all of the different geometric solids behave in one of two ways when they are buried in the earth and subjected to static and dynamic forces from above. Some geometric forms such as the sphere, spheroid, ellipsoids, and hyperboloids have equal ability to support loading above ground or as a buried structure. Other structures which we will classify as "dependent" geometric shapes demonstrate a striking improvement in their ability to sustain loading from above when they are buried in the ground. As buried structures, they are dependent on the surrounding earth for their ability to resist loading from above. A typical example would be a cylinder. When a cylinder is loaded from above and is not surrounded by earth, it will collapse under relatively moderate loading. In contrast, when the same cylinder is buried in the ground and subjected to loading from above, it will sustain considerably greater loads without collapse. The reason for this is that as the top loading tends to decrease the vertical diameter, the

horizontal diameter is slightly increased but is immediately re-
sisted by pressure from the surrounding or supporting earth. Normal
earth will supply considerable resistance to deformation in this
manner and in so doing assists the cylindrical shape to sustain rel-
atively high loading when buried.

In choosing the shape of an underground structure,
it is important that the stresses in it be kept predominantly com-
pressive and, in this respect, bending and buckling stresses be
minimized as much as possible.

Since dependent structures are enhanced in strength
by the surrounding soil, it is important that this factor be taken
advantage of to obtain as much strength as possible from the shelter
design. In the following consideration of specific design possibil-
ities, particular emphasis is placed on dependent forms of structures.

c. Consideration of Specific Designs. In design 1
(Fig. 14), a cylinder halved along its axis is used as the center
roof section. Each end is composed of quarter hemispheres for the
roof. The structure is completed with curved walls that blend into
a flat floor. The end sections are made in one piece and consist
of a quarter spherical roof, curved sides, and a flat floor.
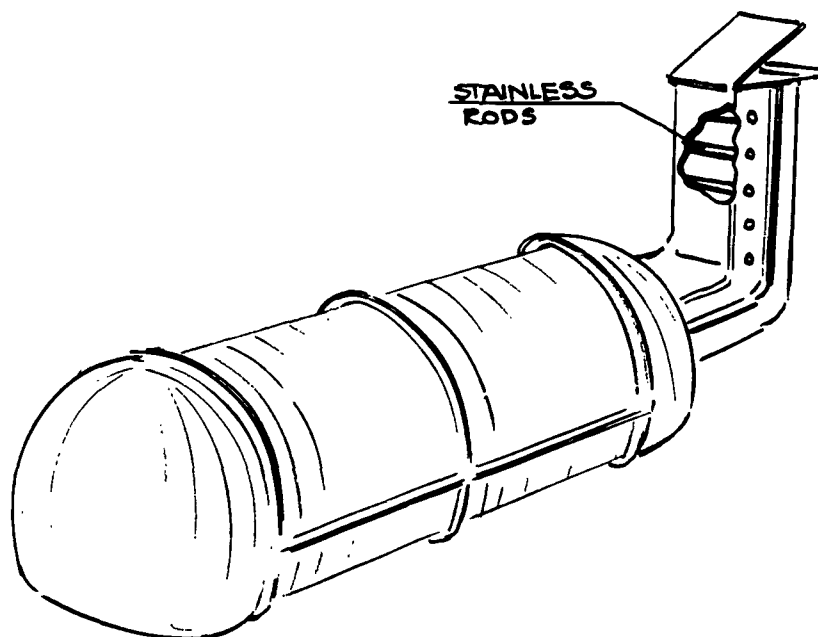


Fig. 14. Design 1.

The various sketches of this design show it to be split up into different size sections for convenience in shipping and storage. The manufacturing process employed will permit these sections to be made as large or as small as desired.

It will be noted from the sketches that the sections are fitted with flanges for assembly. In the latter operation, an epoxy adhesive would be used to coat the inside surfaces of these flanges, and the surfaces would be drawn together employing corrosion-resistant metal bolts and nuts.

The over-all dimensions of a shelter of this design large enough to accommodate 20 people would be 8 feet high, 11 feet wide, and 25 feet long. This is based on providing 80 cubic feet of volume per person.

It will be noted from the cross section view (Fig. 15) that $4\frac{1}{2}$ feet on each side of the centerline a 6-foot head clearance is provided. Reinforced plastic removable seats which can double as bunks could be readily provided to snap into grooved recesses in the side walls. Over these bunks, there is ample head room for sitting. Space beneath these bunks will be valuable as storage of food and sanitary supplies.
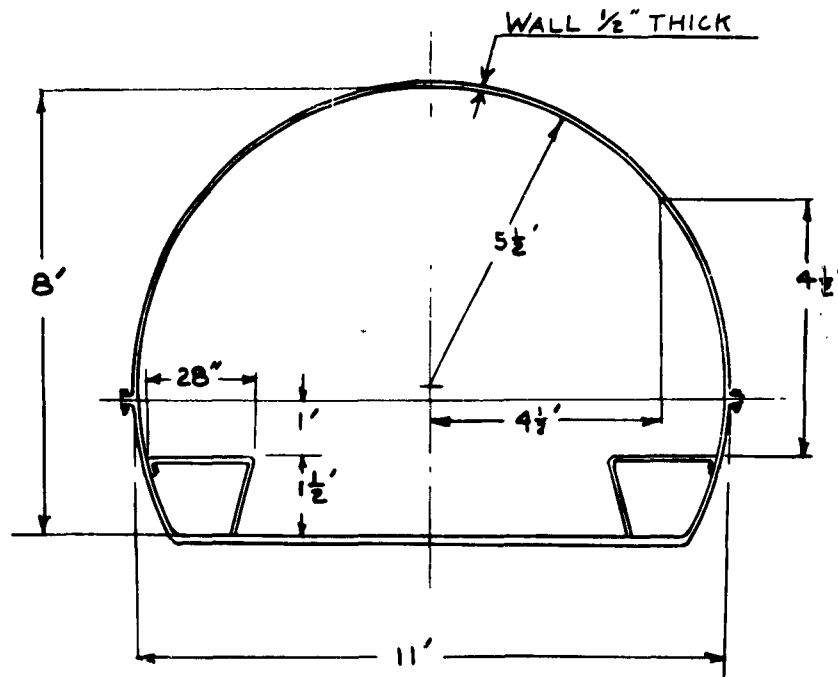


Fig. 15. Cross section view of design 1.

This comparatively long and narrow design will lend itself well to separations of groups such as families, employing fabric curtains hung from the ceiling.

In order to accommodate multiples of 20 people, it is only necessary to provide additional center sections and bolt them into place between the ends.

Figure 16 shows an exploded view of this design picturing the component parts prior to assembly. It also shows the parts given a corrugated type of double curvature to increase their rigidity.
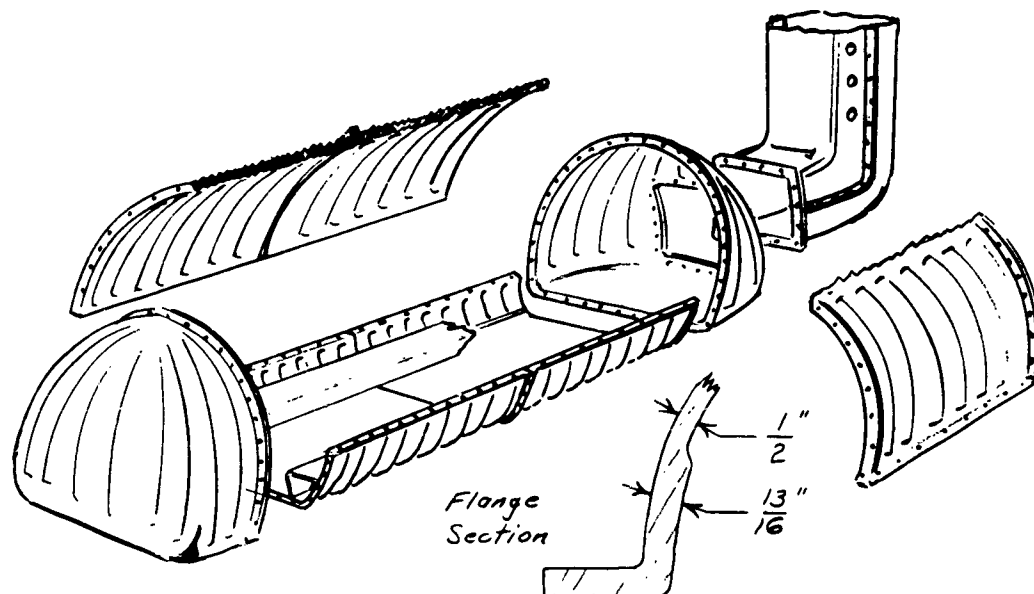


Fig. 16. Exploded view of design 1.

Figure 17 is a side view of the structure assembled underground. It will be noted that the top of the entranceway is installed below ground level to protect it from damage from the blast wave. This entranceway is made from two reinforced plastic pieces which are bolted together at their flanges and then, in turn, bolted to one end of the shelter.

Due to the efficiency of this shape from a structural standpoint and the useability of internal volume, it is one of the most promising of the various designs considered. In making this preference, other factors such as ease of manufacture, shipping, and assembly were considered.

142
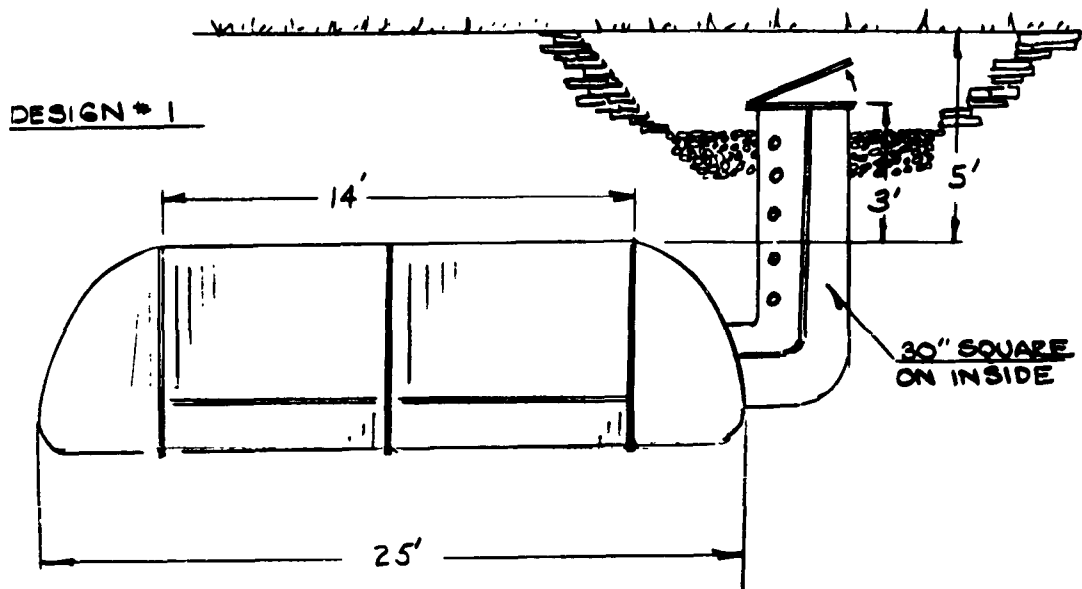
DESIGN # 1



Fig. 17. Side view of structure assembled underground.

Design 2 (shown in Fig. 18) is similar to design 1, except that the half cylindrical roof merges with a short length of vertical wall section. The floor is a flat member, similarly joined to combined roof and wall using vertical flanges. It has been suggested (Reference 1) that additional resistance to blast loading might be achieved in an underground shelter by permitting the side walls of the structure to "punch through" into the earth and in so doing absorb considerable energy. As this energy is actually absorbed and dissipated by compressing soil beneath the side walls of the shelter, this feature would permit the structure to sustain a greater blast loading than would otherwise be possible.

Figure 19 illustrates one manner in which this might be done. At the top is shown a detail of a section of side wall bolted to the floor. In the region of the bolt, the side wall has been deliberately thinned to reduce its shear strength. When the impact blast loading reaches a fixed value, the roof and side walls of the shelter would move downward as the bolt shears through the weakened or thinned area of the wall. This action would permit the bottom of the side wall to punch down into the earth below.

Figure 20 is a design refinement that takes into account the unpredictability of the condition of the earth and its ability to absorb energy through compacting. In this design, we have provided the bottom of the side wall with an extruded strip of
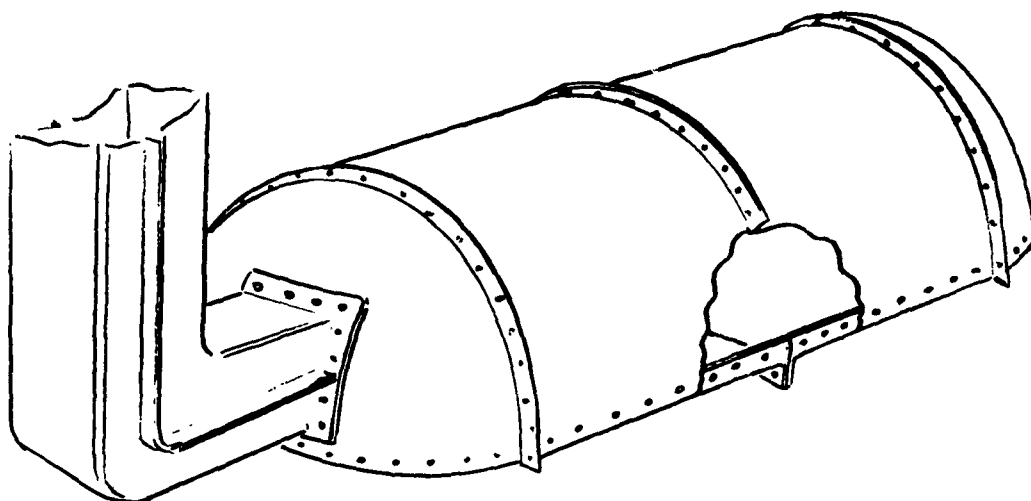
Fig. 18. Design 2.
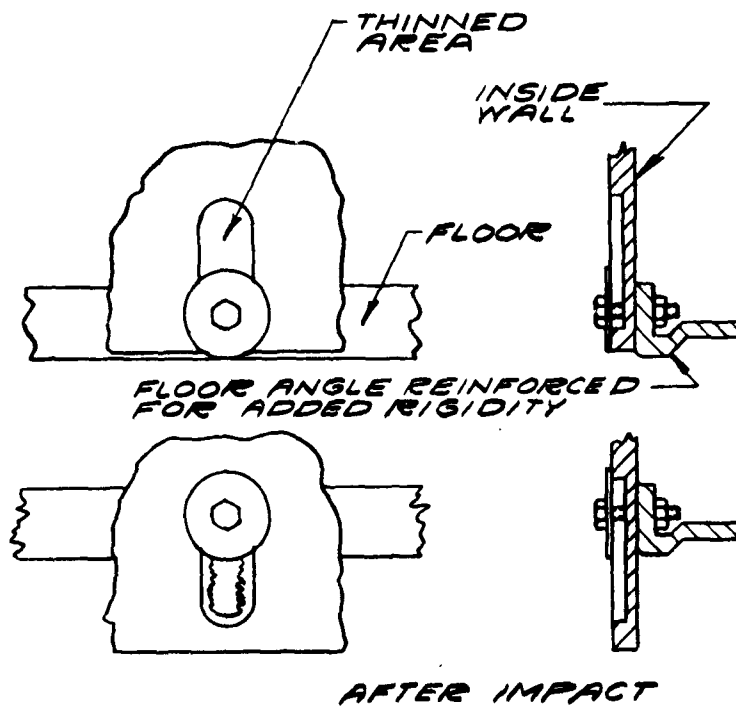


Fig. 19. Movable soil compression feature.

144



RIGID
SKIN
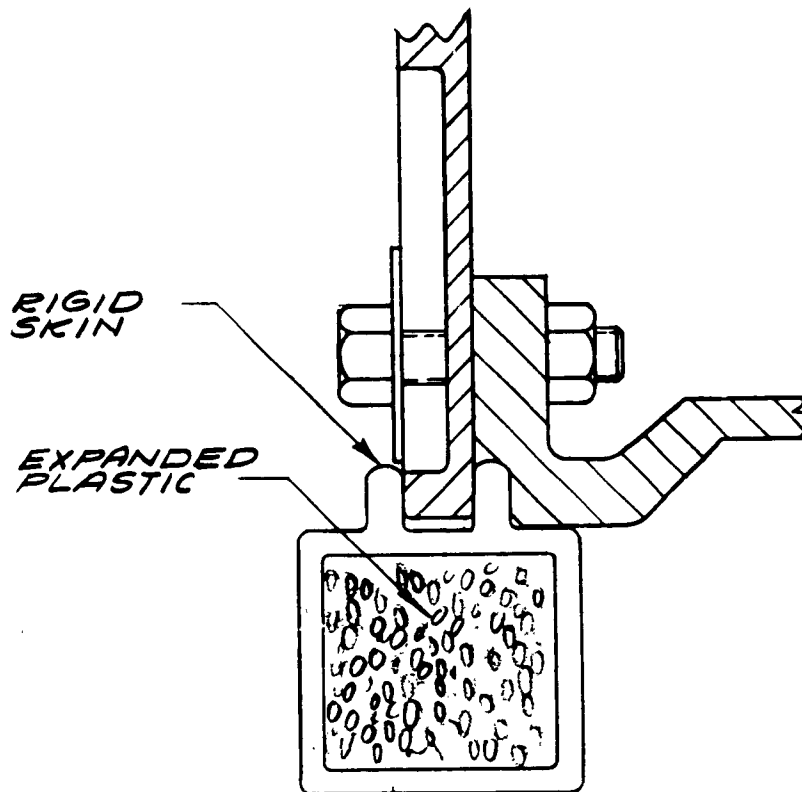
EXPANDED
PLASTIC

Fig. 20.   Refinement of design 2.


flexible plastic material with a horizontal rectangular cavity.
This cavity would be filled with an expanded rigid plastic material
such as polyurethane, with a density chosen to give the desired
energy absorbing characteristics.

In use during high-intensity blast loading, the side
walls would punch into this material, absorbing energy as the frang-
ible plastic was crushed.  This principle has been used in a variety
of unrelated applications where it efficiently dissipates impact
energy.

Figure 21 shows various shapes of roof configurations
that could be used in the two designs of shelters already discussed.
Beginning at the top:  (a) is a semicircular cross section which is
not particularly advantageous from the standpoint of reduced head
room that it gives on each side of the center;  (b) is essentially
design 2 where the head room has been increased by adding short
lengths of vertical walls (these vertical walls would have to be
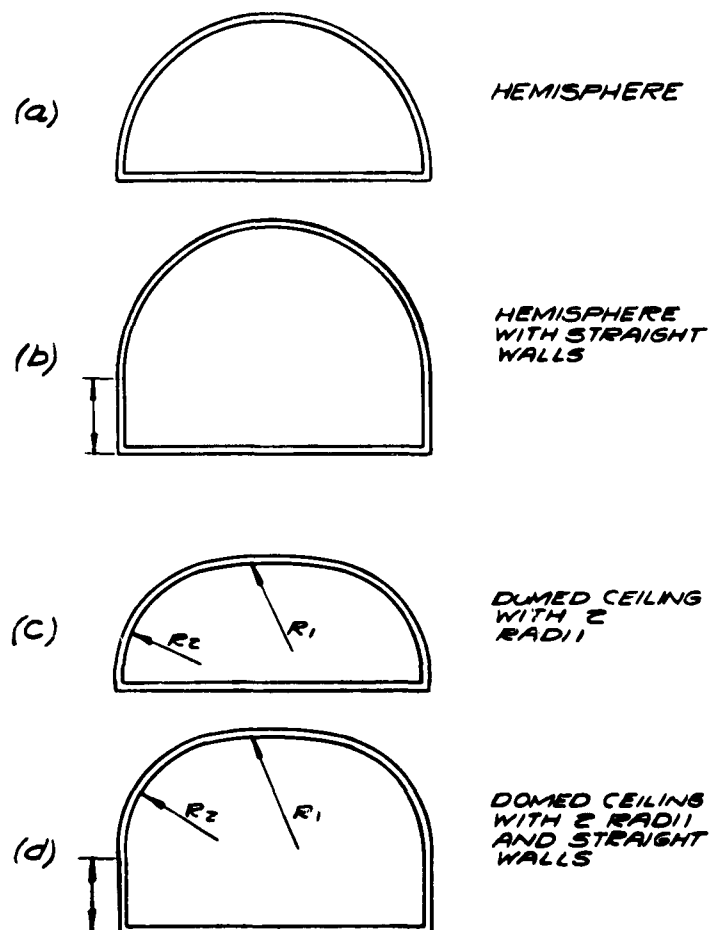
Fig. 21. Possible shelter cross sections.

given some form of double curvature in order to give them added re-
sistance to buckling); (c) illustrates a flattened dome ceiling
formed using two different radii (it has the same disadvantage as
the semicircle in that limited head room exists on each side of the
center; (d) this head room has been increased using vertical walls
(the same objections exist to the use of these walls as are common
to (b) above).

Design 3 (shown in Fig. 22) is formed of sections cut
from spheres. This design is structurally strong but is not as ad-
vantageous from the point of view of useable internal volume. The
design does have the advantage, however, of separating the structure
into family-size units. It would be extremely simple to give greater
privacy to these units through the use of opaque curtains. Another

disadvantage of this design would be the problem of providing it
with bunks which must be in a straight line and not be readily
adapted to the contour of the curved side walls.

Design 4 (Figs. 23 through 25) is a shelter in the
shape of an oblate spheroid. Here again, the design is advantageous
from the standpoint of structural configuration, but the useability
of the internal volume for occupancy is not highly efficient. As in
design 3, there would be the problem of adapting bunks to the curving



**FLOOR**
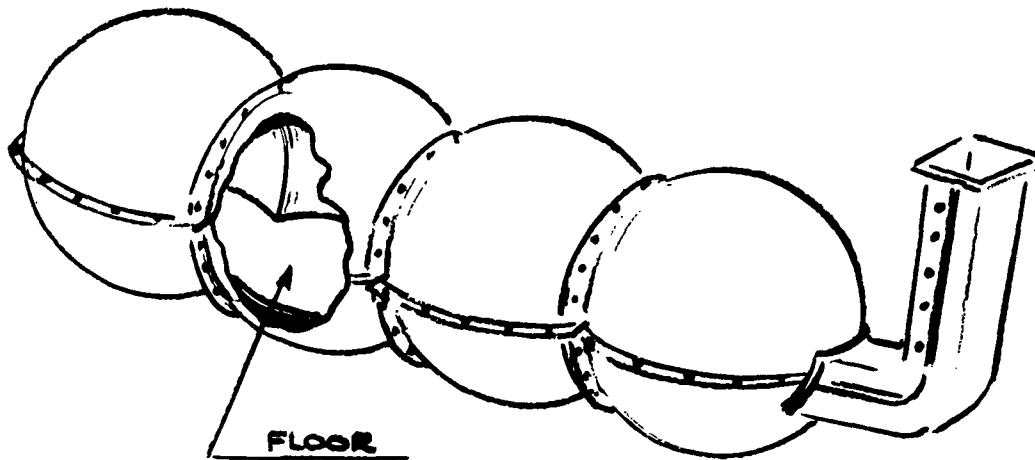
Fig. 22. Design 3.
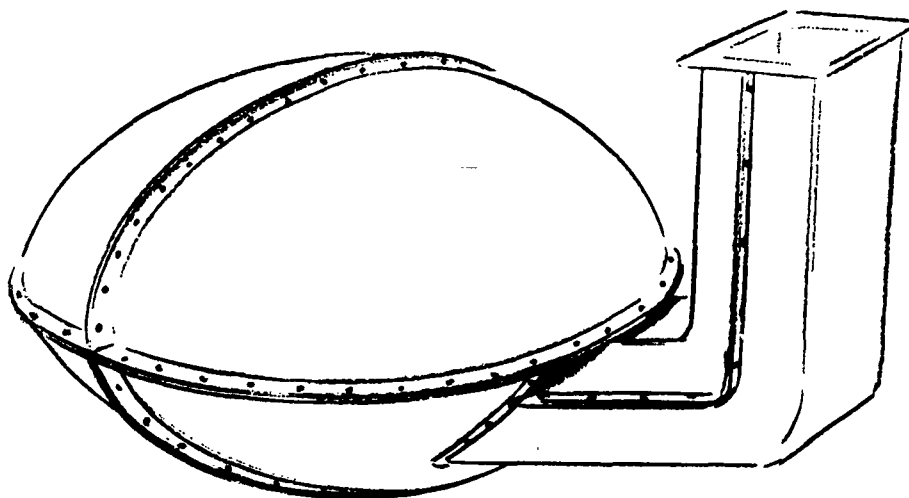


**OBLATE SPHEROID**

Fig. 23. Design 4.
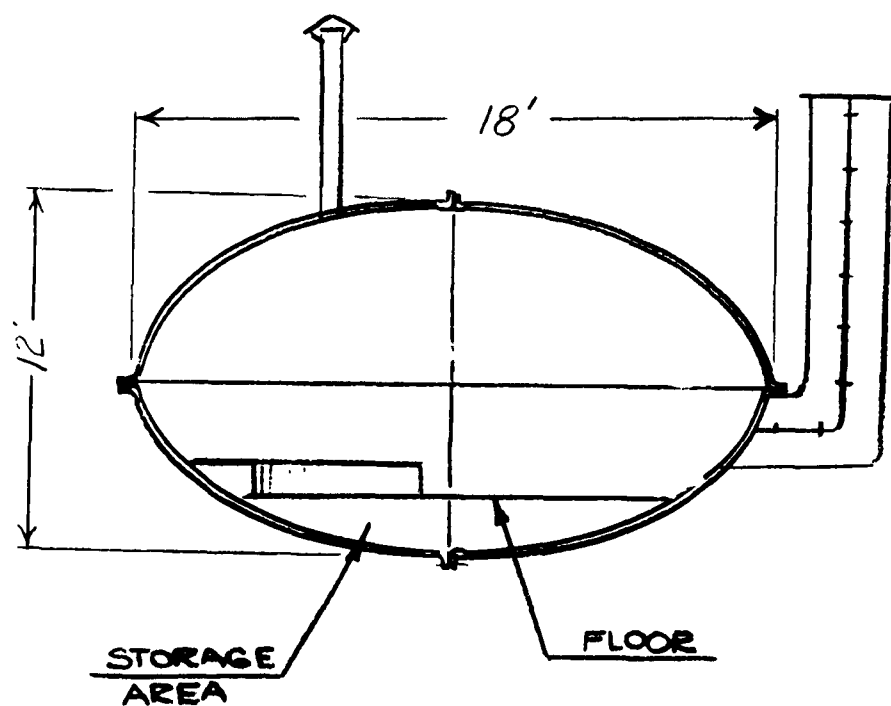
Fig. 24.  Cross section - design 4.
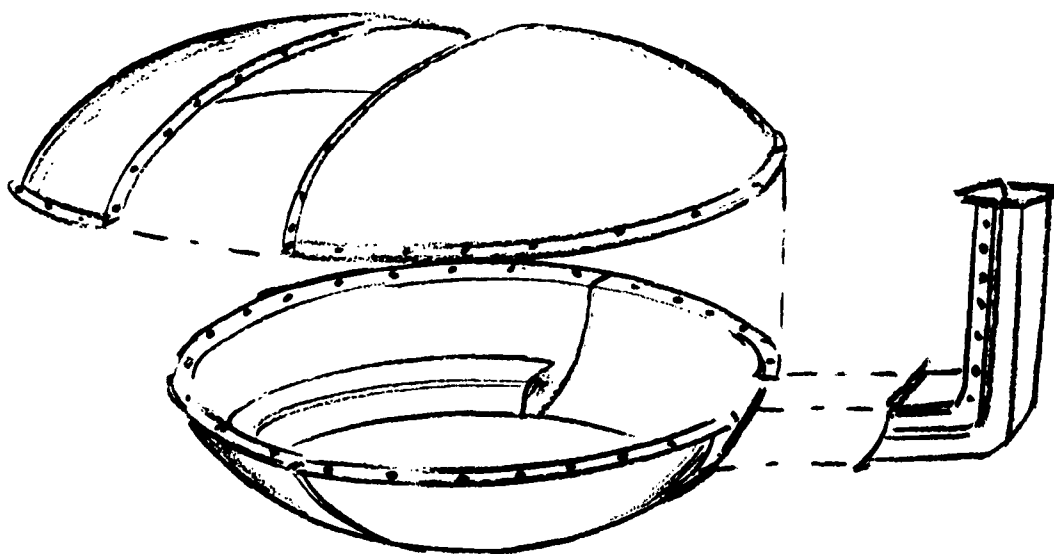


Fig. 25.  Exploded view - design 4.

148

side walls. Reduced head room would exist all around these walls. Considerable head room would be lost due to the necessity of fitting the structure with a relatively flat floor. Such a design would be rather difficult to partition off using curtains to obtain privacy.

Design 5 (Fig. 26) is one that would employ a combination of a comparatively thin shell of reinforced plastic covered by a comparatively heavy layer of steel-reinforced concrete. In this design, the reinforced concrete base would first be poured and the assembled shelter placed over it. An additional reinforced plastic shell, larger in size, could then be lowered over the first to provide an outside form for the concrete.
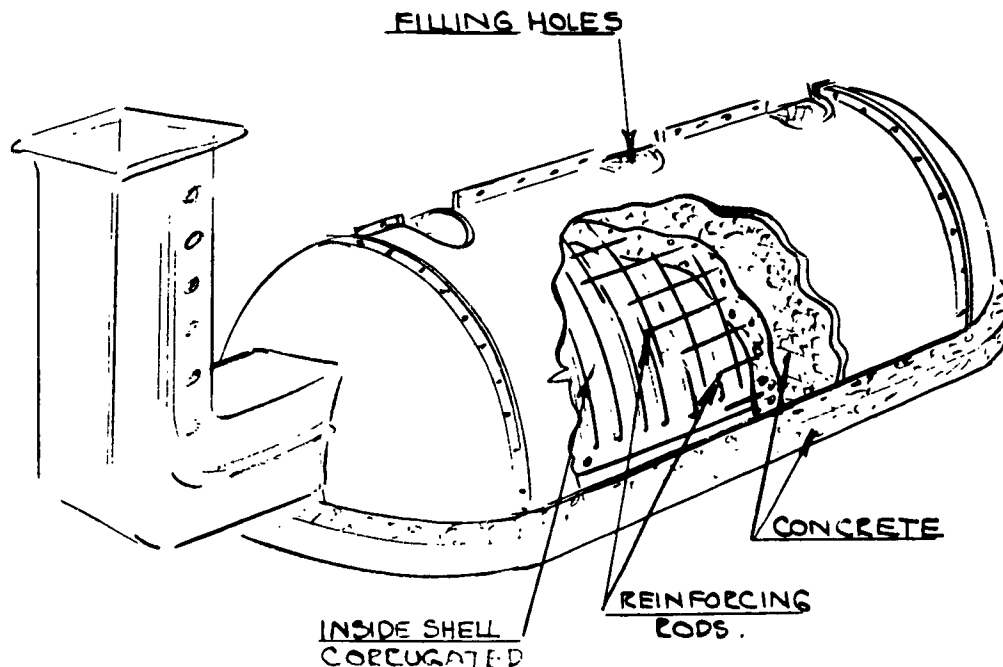


Fig. 26. Design 5.

This outer shell would contain large holes which would be used for charging and compacting the concrete. A transverse cross section is shown in Fig. 27.

After the concrete had set, the outside shell would be removed for reuse as shown in Fig. 28. The remaining shelter would have an internal surface of reinforced plastic with a thick structural shell of reinforced concrete over it. This shelter would give more protection against blast than any other designs of purely reinforced plastic shelters considered.
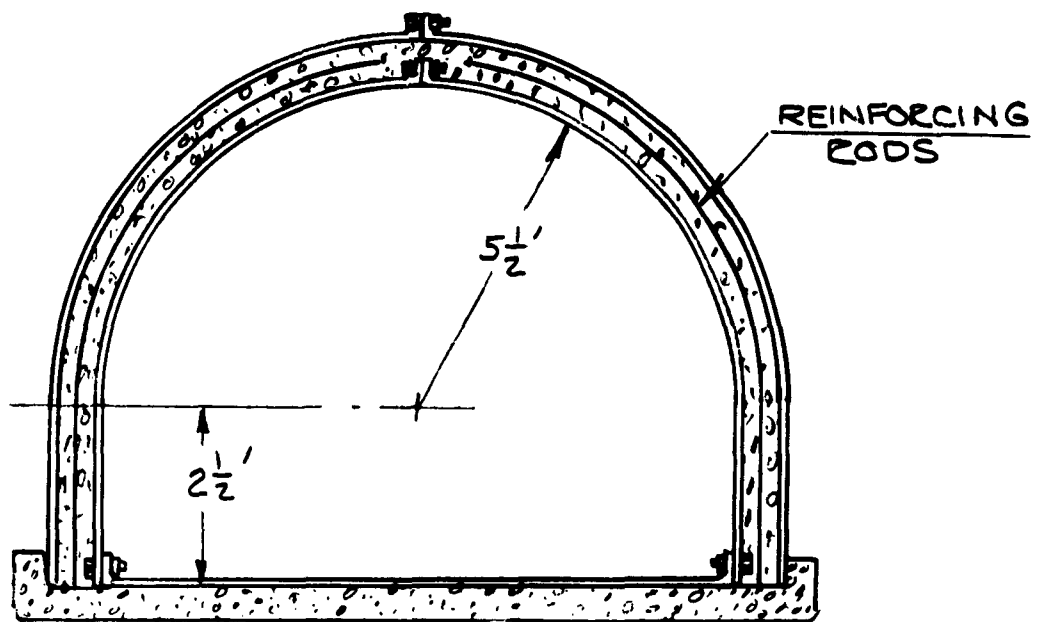
Fig. 27.   Cross section - design 5.
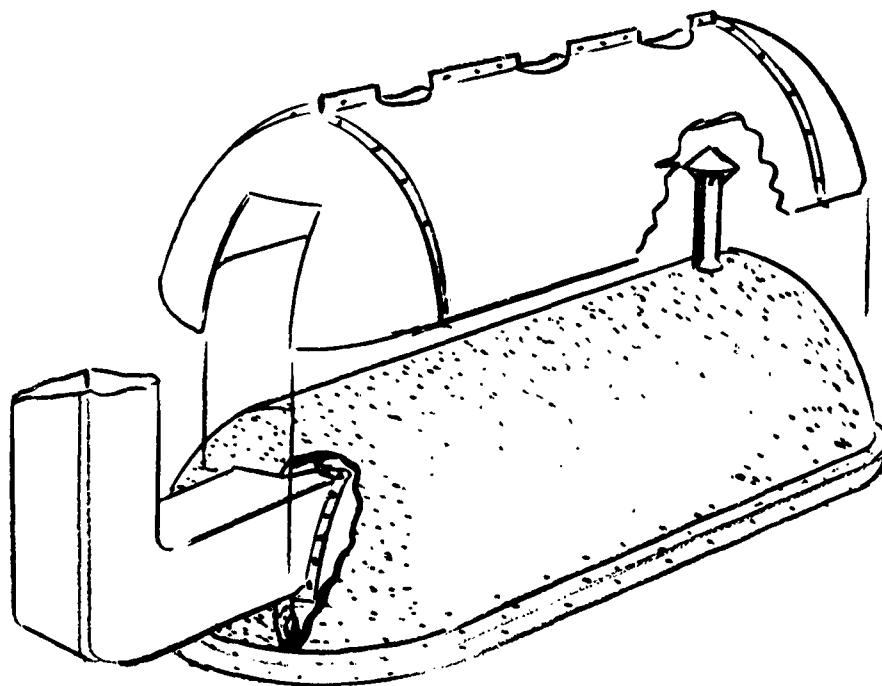


Fig. 28.   Exploded view - design 5.

Reinforced plastic forms for the pouring of concrete are becoming increasingly popular in modern architecture. Other than flat shapes are extremely expensive to duplicate in the materials ordinarily used for concrete forms. An example of such a shape would be the arched roofs that have been considered in the shelter designs. Reinforced plastics are used for this purpose because they constitute the least expensive method of making forms in curved shapes. It should be emphasized that there is considerable precedence in shelter designs to the use of reinforced plastic forms for concrete casting.

In this brief discussion, no attempt has been made to cover all of the possibilities that present themselves as practical shapes that could be made from reinforced plastics. The most important thing to note here is that unlike the design of structures using conventional building materials, the designer of the reinforced plastics shelter is considerably less limited in his consideration of different configurations. To make almost any design of structure from these materials, it is only necessary to provide a rigid male mold cavity and to spray chopped glass and resin onto it simultaneously.

In addition, it is a comparatively simple matter using reinforcing plastics to incorporate into the shelter design numerous other features such as shells, containers, seats, bunks, cavities for waste containers, and even a floor sump for collection of leaking ground water.

9.    Factors to be Considered in Shelter Design and Structural Material Selection.

a.    Long-time Static Loading and Effect of Moisture. Reinforced plastic materials are similar to other structural materials in that they will fail under long-term continuous loading and stresses considerably below the ultimate stress for short-term loading. Continued exposure to moisture, such as would occur in most underground installations, has a similar effect of lowering the ultimate stress of reinforced plastics. These are effects that must be given thorough study in the selection of materials, factors of safety, and design stresses. The effect of this reduction in strength on the selection of factors of safety and design stresses is discussed in detail in the section entitled "Design Calculations."

Figures 29 through 31 give tensile, flexural, and shear long-term loading strengths for chopped glass mat, woven roving, and cloth reinforced polyester laminates with the latter continuously immersed in water (data from Reference 2). These graphs
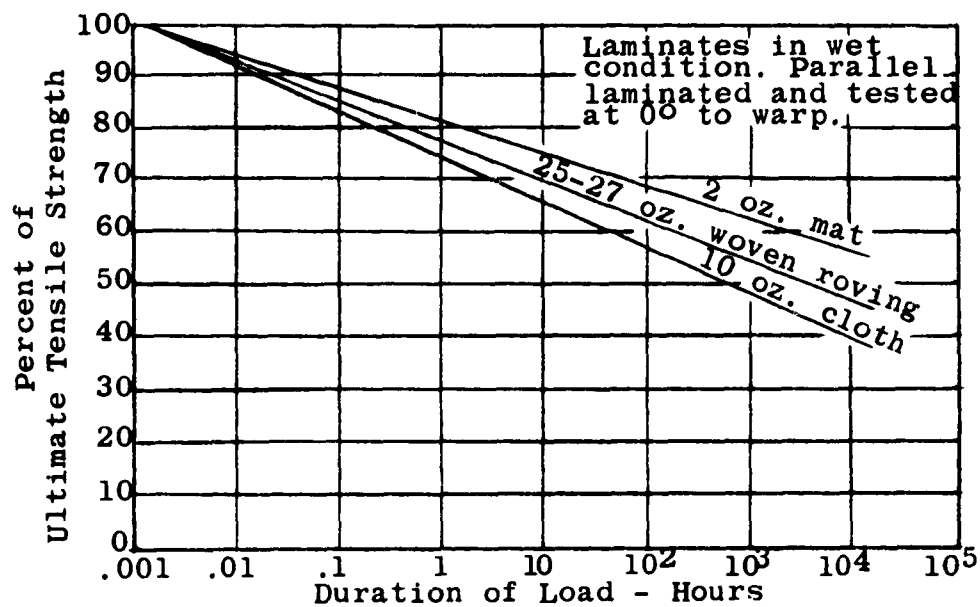
Fig. 29. Tensile strength retention of continuously loaded polyester fiberglass laminates.
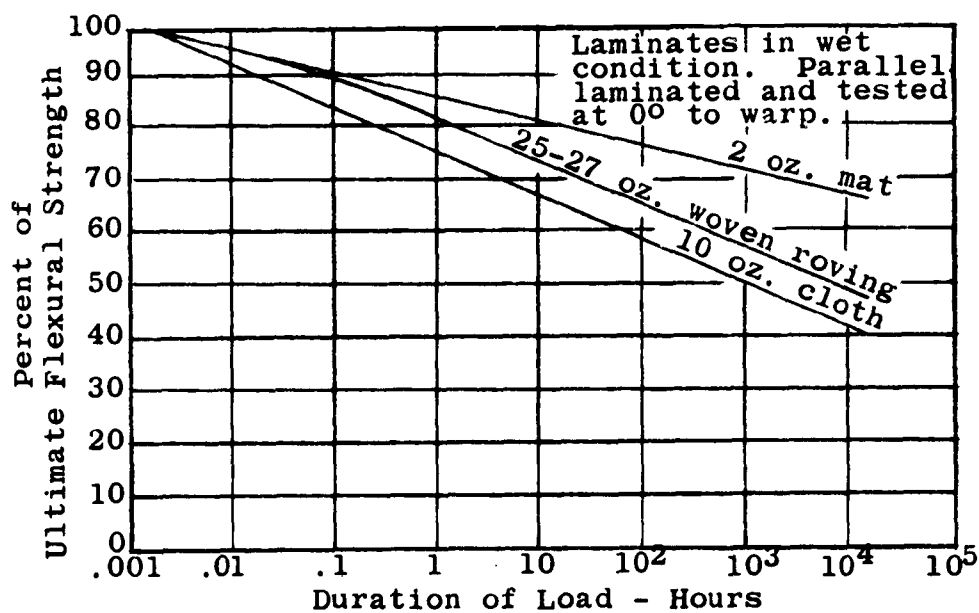


Fig. 30. Flexural strength retention of continuously loaded polyester fiberglass laminates.
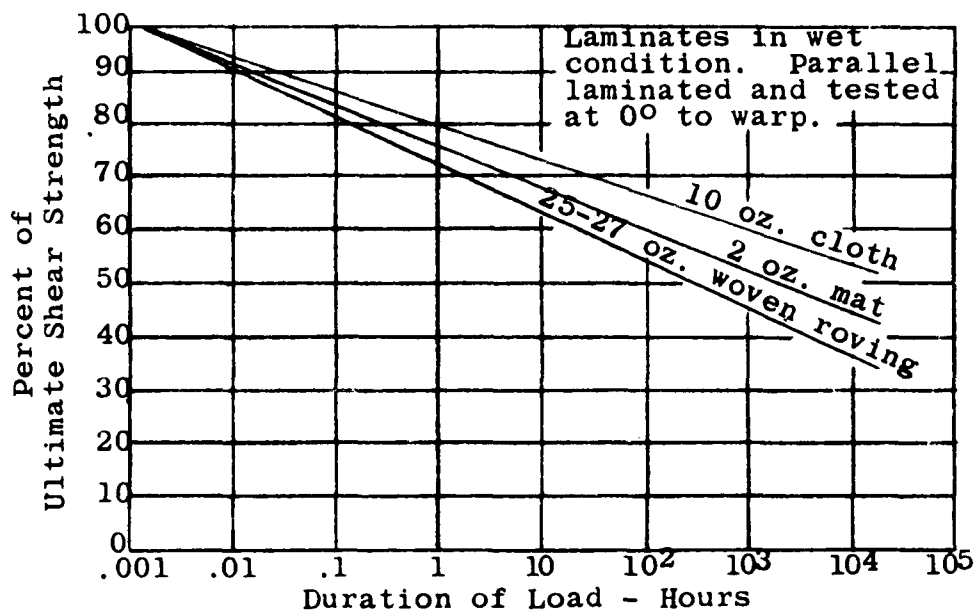
Fig. 31. Shear strength retention of continuously loaded polyester fiberglass laminates.

are important in that they show the percent of short-time ultimate stress at which the laminates will fail after a given time. For example, a 2-ounce mat laminate when continuously loaded at 57 percent of the short-time ultimate tensile strength in contact with water will fail after 10,000 hours, or roughly 1 year. These graphs will be referred to in the Design Calculation section.

The strength of the 2-ounce mat laminate is essentially equivalent to that of a sprayed-up gun laminate provided that the glass contents of the two are similar. It is very interesting to note from these charts that although the mat and gun laminates are considerably less expensive than those made from woven roving or glass cloth, the former is superior in percent of retention of both tensile and flexural strength. These two characteristics are very important to the particular application being studied.

Only in shear strength is the mat laminate inferior to either of the others in strength retention. As areas under high shear strength are limited in the designs, they can be easily reinforced by increasing the wall thickness.

These fundamental considerations verify the correctness of choice of chopped glass strands as a reinforcing material.

153

b.   __Blast Loading.__  It was suggested that blast loading be treated as a static force as a matter of simplicity in preliminary design considerations.  It is felt that this is a very practical approach to an already complex design problem.  However, it should be kept in mind that this assumption like all assumptions should be evaluated when experimental structures become available for testing.  Since the thermonuclear blast can raise the atmospheric pressure this additional 20 psi in less than 1 millisecond, the effect of this suddenly applied shock loading on structural members could have a far different effect than application of a static overload for a short period of time.

Much has been theorized and postulated concerning the effect of dynamic blast loading on various structures with shapes dependent on surrounding earth for support.  Reference 1 develops in considerable detail the subject of dynamic loading on such dependent structures and the soil around them.  It emphasizes that the response of dependent structures cannot be defined unless the soil's resistance to lateral motion is known.  It is pointed out that this reaction will vary considerably between soils that exhibit different degrees of cohesion.  These soils differ markedly in their Poisson's ratio which is the ratio of lateral unit strain to longitudinal unit strain during loading.

This referenced study also considers the modulus of elasticity of the soil, obviously a factor in the dynamic loading of the soil and gives a 100,000-psi figure for seismic loading.  This figure would be a valid one to employ for blast pressure loading.  In comparison, a modulus of 2,000 psi has been determined for a slow rate of loading.

This points up a very likely safety factor in considering blast loading to be a static condition.  However, so many dubious assumptions have to be made in calculating soil reaction to dynamic loading and the effect on a dependent structure that the results of any calculations of stresses produced in the structure itself may not be representative of any actual condition likely to occur.

One very important consideration is the difference to be expected in the protection from blast of earth dependent structures depending on the manner in which they are installed.  For the fully buried structure, it is evident that the surrounding earth will increase the top loading which it will sustain in comparison to what it would support if it were installed on the surface.

In some areas of our country where the ground water level is so close to the surface that the shelter cannot be buried, it will have to be mounted on the surface and covered with a mound of earth large enough to give the 3- to 5-foot overburden. As the amount of support given the sides of the structure will be considerably different and as the blast wave is likely to strike one side of the structure before the other, reinforced plastic shelters of a dependent configuration will offer considerably less protection from blast loading if they are not installed below ground.

In locations where ground water level approaches the surface, concrete is to be preferred in blast-resistant shelters.

c. <u>High Ground Water Level</u>. In Florida, for instance, normal ground water level can be as high as 1 or 2 feet below the surface. This is a constant condition. In many other areas of the country where water tends to collect during rainy seasons in low areas, the ground water level may approach the surface at different times of the year.

If the ground water level rises until it is several feet above the bottom of the shelter and the weight of the water that would occupy the shelter at this level is greater than the weight of the shelter itself, the shelter will tend to "float" upward.

Natural movement that takes place will depend on the extent of this buoyant force and the softness of the earth above the shelter. Already, with the limited number of family shelters made from reinforced plastics and metals that have been installed, a sizable amount of difficulty has occurred where this factor was not taken into consideration before installation. Buried shelters have been known to float upward several feet due to the buoyant force of high ground water level.

Even though the ground water level does not rise sufficiently around a shelter to cause difficulty through a buoyant effect, it can be extremely troublesome if the shelter is not watertight.

d. <u>Ground Water Seepage</u>. From the difficulties that home owners frequently have over most of this country from water seepage into basements made from concrete block and cast concrete, it will be evident that the problem will be even more serious in a shelter which will normally have its floor at a lower level. A cavity filled with water to 1 foot above the bottom level of the shelter will exert a pressure of approximately 62 pounds per square

foot in its effort to enter the shelter. A water level 5 feet above this surface will exert five times this pressure or about 310 pounds per square foot.

Where a shelter made from any material is assembled from components, ground water leakage is likely to be a serious problem regardless of materials used in the shelter. If the assembly is done by individuals who are inexperienced in the techniques, the possibility of leakage will be increased.

If a considerable amount of water is allowed to collect in a community shelter before use, it will obviously be of limited value in an emergency. Therefore, not only will the location for installation have to be chosen carefully to limit difficulties with high ground water level, but it is essential to provide either a completely watertight structure or an automatic sump pump to keep it dry.

Investigation of this matter has shown it to be an extremely important consideration that will to a large extent determine the usefulness of community shelter installations.

For occupancy, it is not only necessary to keep the shelter substantially free of water but it is also desirable to keep the internal air temperature as close to normal room temperature as possible.

e.   Thermal Insulation. Unlike steel, aluminum, or concrete, reinforced plastics are poor conductors of heat. In a test conducted during the winter in Philadelphia under 3 feet of frozen soil a reinforced plastic shelter having a wall thickness of 3/8 inch was found to have an air temperature of 80° F. This was recorded during a trial with the shelter containing occupants. Depending on the volume of ventilation, the temperature could be controlled at a lower level without difficulty.

It has been reported by family shelter manufacturers that the heat insulation provided by a 3/8-inch wall is sufficient to prevent condensation of moisture on the inside surface of a buried shelter even during summer periods of high humidity. The temperature problem with reinforced plastic shelters will exist during the hot summer when sufficient ventilation must be provided to prevent temperature rise above a safe level.

10. <u>Design Calculations</u>.

     a.   <u>Stresses Involved</u>. For consideration of stresses, the basic shape of design 1 is used as an example and consideration is confined largely to the roof section which is essentially a cylinder halved along its axis, with quarter spherical ends. After the shelter has been installed in a cavity in the ground dug with a backhoe or bulldozer, the next step will be to backfill the earth around and above it. If the earth contains sharp rocks and this backfilling is done with a bulldozer or tractor, the roof of the shelter will be subject to impact stresses. Unless something is known about the exact nature of these stresses, no meaningful calculations can be made. However, the ability of large boats with reinforced plastic hulls to sustain repeated impacts of high magnitude without failure is indicative of the performance to be expected by reinforced plastic shelters.

     After the shelter has been buried, it will be subject to several types of constant stress from the earth cover.

     (1)  <u>Compressive Stress</u>. In a shelter of this design, the weight of the earth above, coupled with the restraining effect of the earth on all sides will create a compressive stress throughout the structure. This type of stress dominates in importance.

     (2)  <u>Shear Stress</u>. Referring to design 1 (Fig. 14), shear stress will assume a maximum at the location where the flange joins the circular side wall. For this reason, the stress will be calculated only at this point.

     (3)  <u>Buckling Stress</u>. Buckling stress is the elastic stability of the structure under loading or its tendency to fail by buckling. The maximum tendency to buckle under loading would naturally occur in shelter designs with flat vertical walls.

     In any underground structure such as the type being considered, there is a variety of different types of stresses present at the same time throughout the structure. It is important to know which are the governing stresses and at what locations they can be expected to reach their maximum. It is equally important to know a safe working level for each stress.

     (4)  <u>Design Stresses</u>. As previously mentioned, tables of physical values for plastic materials, like

building materials, give ultimate strength values generally obtained with short-time tests. To obtain a safe design working stress, the designer divides these values by a factor of safety. Normal factor of safety used with reinforced plastics where failure of the item does not affect human life is 4. For the chosen shelter design, a factor of safety of 5 was used.

Such an assumption should not be made without reasonable justification taken from test values that duplicate the additions of constant loading and constant exposure to water which are common to this application. Figures 29 through 31 are graphs showing the decrease in ultimate strengths of wet constantly loaded specimens with respect to time.

It will be noted from these graphs that $10^4$ hours is approximately 1 year and $10^5$ hours, 10 years. If the line representing the 2-ounce mat is taken as comparable to the sprayed laminate and is extrapolated to the $10^5$ line, in 10 years the tensile strength is reduced to 50 percent; the flexural, 62 percent, and the shear strength, about 37 percent. It is evident then that taking a working stress that is 20 percent of the ultimate stress (using a factor of safety of 5) is a reasonably safe assumption providing that other factors that should be considered are not overlooked.

The first of these factors is the rather large variation in strength that can exist in different areas of a single reinforced plastic structure due to production variables. There is also another important factor that is not ordinarily considered by the designer calculating wall thicknesses required for structural soundness.

The physical values given for the ultimate stress of plastics are often run on samples 1/8 inch thick. If, for instance, working stresses derived from these are used in calculating the thicknesses of members approximately 6 times this wall section, the error involved can be about 22 percent. The reason for this is that thin sections exhibit a greater strength in pounds per square inch than do heavier sections of the same material.

b. **Applicability of Design Calculations.** It will thus be evident that calculation of wall thicknesses of structures such as reinforced plastic underground shelters involves considerably more than substituting values taken from physical tables into handbook formulas. Considerable attention must be given to: (1) What these values actually are after a long period of loading under simulated conditions and (2) the basic limitations of the methods of testing and the meanings of the values themselves. In addition to

this, any formula used in calculating stresses and wall thicknesses must be cautiously examined from the light of the assumptions that are made in its derivation, its practical value, and the interrelation between the stress considered and other stresses existing simultaneously in the structure.

The values obtained from such formulas cannot be taken as absolute values that can be used without question to produce a completely reliable design, but must be carefully weighed in the light of the practical experience that has been accumulated to date. This is the procedure that will be followed in the use of illustrative calculations that follow.

c. Calculations of Structure Thickness. As the stresses on the buried dependent enclosure are primarily compressive, the thickness required will first be calculated considering compressive stress alone. Since the earth overburden will exert approximately 4-psi static pressure and an overpressure of an additional 20 psi is being considered, a total pressure of 24 psi is used.

The ultimate compressive strength for the spray-up laminate is 16,000 psi minimum. Dividing this by a factor of safety of 5 gives a working compressive stress of 3,200 psi.

(1) Calculation of Wall Thickness Based on Compressive Stress. The formula used below assumes a uniform hydrostatic pressure on the outside of the semi-cylindrical arch, with the extremities of the arch secured with hinges.

$$S_c = \frac{P_c R}{t}$$

$S_c$ = 3,200-psi compressive working stresses

$$t = \frac{P_c R}{S_c}$$

$P_c$ = 24-psi hydraulic pressure

$$t = \frac{24 \times 5\text{-}1/2 \times 12}{3,200} = 0.495 \text{ inch}$$

R = 66 inches, the radius of our design 1

or roughly 1/2-inch thickness

t = wall thickness in inches

(2) Calculation of Flange Thickness Based on Shear Stress. The thickness required in the flange where it joins the semi-cylindrical roof at each side of the section shown in Fig. 27 is now calculated.

If an arch 1-foot wide with 24-psi vertical pressure being exerted on a projected area 11 feet by 1 foot wide is assumed, the total force is 38,000 pounds divided between the two sides of the shelter arch. Dividing by 2, the force equals 19,000 pounds exerted on each 1-foot-wide flange.

If the shearing force acts on a 1-foot length, the thickness of the flange, the corresponding stress is:

$$S = \frac{19,000 \text{ lb}}{12'' \times t''} \text{ where t is flange thickness}$$

Taking a maximum shearing stress of 9,900 psi, reduced by a safety factor of 5 to 1,980 psi, the flange thickness becomes

$$t = \frac{19,900}{12 \times S} = \frac{19,000}{12 \times 1,980} = .80 \text{ inch.}$$

Computation based on consideration of the elastic stability of thin-walled structures of large radii indicates a need for much heavier wall sections when the support of the soil is neglected. In dependent structures, the soil responds to deflections preceding instability with forces resisting further deflection. While the response of the soil varies markedly according to soil constitution, the net effect is in the direction of counter-balancing the relatively small forces needed for unstable failure of thin-walled structures. It is assumed, therefore, that the bending stresses may be neglected as subsidiary in importance to the primary compressive and shear stresses.

d. **Comparison of Design Calculations with Actual Field Experience.** One of the larger manufacturers of plastic family shelters has a design made with the same material and employing the same process contemplated for design 1. This design (Reference 3) differs in that the cross section is essentially square with rounded corners and the internal width is only 6-1/3 feet. These two factors would tend to offset each other somewhat.

The flat roof would require more material than the arched design contemplated, but the narrower span would normally require less material. Although the designs are not directly comparable, they are similar enough that the use and test experience already gathered from this shelter should be considered. Using the model that has a 1/2-inch wall thickness, static loading tests were

run for 4 years employing a 600-pound-per-square-foot top loading without serious deflection or failure.

These test results on an actual shelter tend to verify the reasonableness of comparative simple calculations of wall thickness. Thicknesses of from 3/8 to 1/2 inch have become relatively standard for family-type buried shelters made from reinforced plastics.

The No. 1 design shelter should, therefore, have a wall thickness of 1/2 inch except in the region of the flanges where the thickness should be at least 3/4 inch. Using the spray-up gun, it will be a comparatively simple matter to provide the additional thickness in these limited areas and to blend evenly between them.

As previously mentioned, some form of double curvature, such as corrugation of the curved surface of the structure, is recommended for a cylindrical shelter to obtain maximum rigidity. As comparatively little is known about the performance of doubly curved surfaces in reinforced plastics, the exact thickness of corrugated surface that would be required cannot be calculated with accuracy. However, as double curvature increases the stiffness and stability of the structure, it is possible to use a thinner wall section for equivalent stiffness and elastic stability. If for cost and weight calculations, it is assumed that the same amount of material will be used in the corrugated or doubly curved form of the structure, results will be high.

It has been stressed that the exact performance of any calculated design to underground blast loading is dependent on the reaction of the earth around the structure. The matter of soil reaction and the interaction between the soil and the structure is a complex matter which cannot now be completely defined. It is not felt possible to predict the exact overpressure that a buried shelter design will sustain.

However, a shelter buried under 3 to 5 feet of earth that sustains a 20-psi static overload for a prolonged period should certainly be expected to successfully resist a 20-psi dynamic overpressure for a period of several seconds. Long-time static loading of reinforced plastic structures produces failure at stress values considerably lower than is the case with short-term dynamic loading.

Now that, for illustrative purposes, a promising design has been chosen and the wall thickness of material to be used in it has been calculated, a rough estimate of its manufacturing cost can be made.

11. **Estimated Manufacturing Costs of Typical Shelter Designs.**
Table XV gives dimensions, weights, and estimated prices for the
manufacture of designs 1 through 8 considered in this appendix. The
basic shelter assembly has been designed for occupancy by 20 people.
All of the estimated prices given in the table exclude trim, hard-
ware, and accessories.

Designs 1 through 5 are quoted to be made by the "spray-up"
process, 6 and 7 by roving winding, and 8 by contact molding using
hand lay-up. No part of any of these designs exceeds 800 pounds in
weight. This limitation was employed so that the pieces could be
handled readily by a team of six men.

Referring to Table XV, the internal volume given on line 1
is the total inside volume of the erected shelter not including the
inside of the entryway. The internal dimensions given on line 2 of
the table represent the maximum inside width, height, and length of
the shelter proper and do not include the entryway.

The total weight given in line 3 includes only the rein-
forced plastic shelter sections and excludes the entryway assembly
hardware and any concrete that might be employed in the design. The
estimated selling prices in line 4 are also for the reinforced plas-
tic sections of the shelter itself with all designs excluding the
entryway.

For designs 6 and 7, the estimated weights and prices in-
clude ends made from reinforced plastic although they could be made
from a number of conventional building materials. Design 8 is a
sectional roof dome of reinforced plastic. As the floor, walls, and
ends of any shelter employing it would be made from one of a number
of materials such as concrete, the weight, price, and price per
occupant given cover only this roof unit and do not apply to the
complete shelter.

In the next-to-last horizontal line of the table, the cost
per occupant for the first five designs represents the selling price
for the complete shelter divided by the number of occupants, usually
20. In the last line, figures are given for the cost of adding
occupancy for 20 additional people to the 20 already housed by the
units covered above. In other words, the added units would be iden-
tical to the complete shelter except that they would not require an
additional set of ends.

Design 3 (Fig. 22) formed from connected truncated spheres
consists of six spherical units, each with a radius of 4 feet. The
minimum ceiling height between units is 6 feet.

Table XV. Cost, Weight and Dimensions of Shelter Designs

| Item | Design Number | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
| Internal Volume (cu ft) | 1600 | 1600 | 1548 | 1600 | 1600 | - | - | - |
| Internal Dimensions (ft) | 11x8x25 | 11x8x25 | 8x8x45 | 12x18x12 | 11x8x25 | 12x8x20 | 10 dia. x 24 | 10x10x8 (ea dome) |
| Total Weight (lb) | 3600 | 3600 | 3700 | 3100 | 2700 | 3600 | 4000 | 400 per section |
| Estimated Selling Price | $2370 | $2370 | $2440 | $2040 | $2500* | $2550 | $2750 for 24 people | $300 per dome for 10 people |
| Cost Per Occupant | $120 | $120 | $122 | $102 | $125 | $127 | $115 ea for 24 people | $20 per dome only |
| Estimated Price for Added Units for 20 People | $1150 | $1150 | $2350 | $2050 | $2350 | $1500 | $1700 | $600 for domes only |

* Includes cost of concrete and reinforcing rods as material only.

For design 5 (Figs. 26 through 28) which consists of a comparatively thin reinforced plastic shelter inside a concrete exterior, an 8-inch layer of reinforced concrete was included in the design. This layer contains No. 5 steel reinforcing rods (0.625 inch in diameter) priced at $10.00 per hundred pounds with an 8-inch spacing between rods. $500 worth of concrete at $15.00 per cubic yard was used in the cost estimate.

12. **Comparison of Calculated Selling Price of Comparable Enclosures Made from Steel and Reinforced Plastic.** In order to obtain some direct comparison of selling price of a reinforced plastic shelter with price of an identical unit that would be manufactured from steel plate, a price of a typical steel tank with dimensions similar to those required in our shelter was obtained. This tank is a 23-foot-long cylinder 10-1/2 feet in diameter with convex ends giving a total additional length of 18 inches. It is fabricated from 5/16-inch-thick steel, weighs 10,800 pounds, and sells for $1,600 (Reference 4).

Manufacturing cost summary for a reinforced plastic tank having a 1/2-inch thickness and otherwise identical dimensions is as follows:

| | |
|---|---|
| Material Cost | $1,193 |
| Direct Labor | 310 |
| Manufacturing Burden (150%) | 465 |
| Scrap Allowance (5%) | 74 |
| Profit (5 cents per lb) | 168 |
| Total Selling Price | $2,210 |
| for a 3,360-pound Tank | |

The difference in price between the two structures is $610 with a weight difference of 7,440 pounds. Using the applicable shipping cost of $1.00 per mile for 10,000 pounds, we can see that if the two tanks are shipped a distance of 700 miles, they will be equal in their total price plus shipping cost. Beyond 700 miles, the reinforced plastic tank, or a shelter of comparable weight, will have an economic advantage.

It will also be apparent that if the sections of a steel shelter are of identical size to those of a plastic shelter in the designs considered, they cannot be handled without special equipment. If the sections are the same size, they will be three times as heavy when made of steel. If they are made of steel in equivalent weight to the plastic sections, then three times as many sections will be involved and the price of bolts, gaskets, and flanges and the installation time will be considerably greater.

Although the price given for the steel tank includes an exterior coating of asphalt to protect against corrosion, it will undoubtedly be necessary to give this surface a coating of a heat insulating material. This will further increase the price of a steel shelter. As previously pointed out, the reinforced plastic shelter will have more than enough heat insulating properties to produce a comfortable temperature in the interior.

It should be emphasized that the above estimate for a reinforced plastic tank shelter is for a small annual production volume, about 2,000 units per year. All operations in the manufacture of the shelter have been estimated as manual.

Substantial price reduction could be achieved by manufacturing the shelters in large volume. The extent of this reduction is difficult to predict with accuracy without knowing all of the design details and other factors. However, for an annual volume of 20,000 units per source, it is likely that the spraying operations could be mechanized saving about one-fourth of the labor. At this level of output for a single producer, it is probable that the price of glass would drop from 40 to 37 cents and resin from 34 to about 22 cents per pound. Such economies might reduce the price spread between comparative examples in steel and plastic to about $200 per unit.

However, in the above comparison, the tank shelter is estimated as being made completely by hand as compared with a production method that has been employed by the steel fabricating industry for years. This obviously places the plastic product at a distinct disadvantage in such a comparison. It is not known to what degree the price of a steel shelter could be reduced if it were made in annual quantities of 20,000 per source, but it is safe to assume that the savings expected from large volume production would be greater on a percentage basis for the reinforced plastic shelter.

A steel shelter was taken for comparison since steel is felt to be the only material that would compete economically for this application.

13. Use of Concrete with Reinforced Plastic Shelter Components. Under the consideration of design 5 which employs reinforced plastic forms for the casting of a concrete shelter, one combination of these two different materials has already been considered. Due to the fact that these two very different material combinations supplement each other in properties, it appears desirable to consider other instances in which they can be combined to particular advantage.

The serious difficulty of "floating" which could result from buoyancy when the water table rises to a level above the bottom of a shelter has been described. A method of overcoming this buoyancy effect employed by one manufacturer of reinforced plastic home shelters is illustrated in Figs. 32 and 33. As shown in these
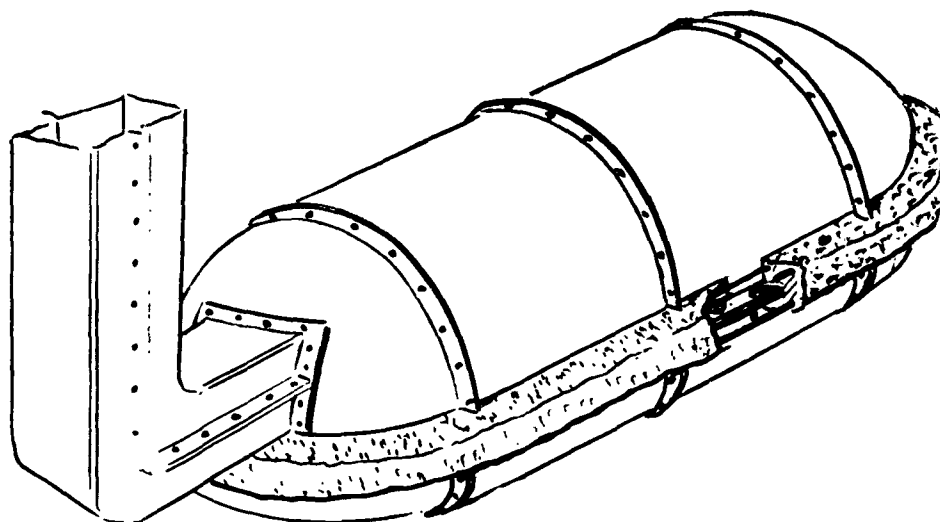


Fig. 32. Structural frame of steel reinforced concrete cast around horizontal assembly flange to overcome floating effect.
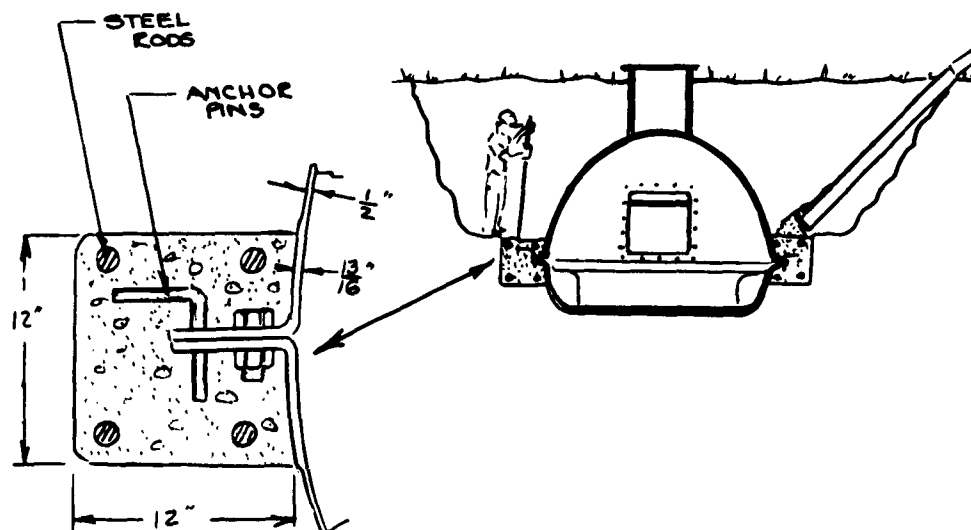


Fig. 33. Method of attachment of frame shown in Fig. 32.

sketches, a complete continulus rigid structural frame of steel-reinforced concrete is cast around the horizontal assembly flange of design 1. Short-angled lengths of steel reinforcing rod join this reinforced-concrete frame rigidly to the shelter.

Such a frame has three distinct advantages:

a. The added weight provided by the reinforced concrete frame tends to overcome buoyancy.

b. The location of the frame around the horizontal joint prevents ground water leakage at this joint.

c. The rigidity and inherent strength of the frame would add measurably to the blast resistance of the entire structure.

It is felt that this approach to the buoyancy problem should receive serious consideration due to its practicability.

It may also be quite feasible to consider a shelter having walls and floor of some conventional material such as concrete but with a roof formed of domes molded from reinforced plastic as shown in Fig. 34. Figure 35 shows a longitudinal cross section of such a construction.
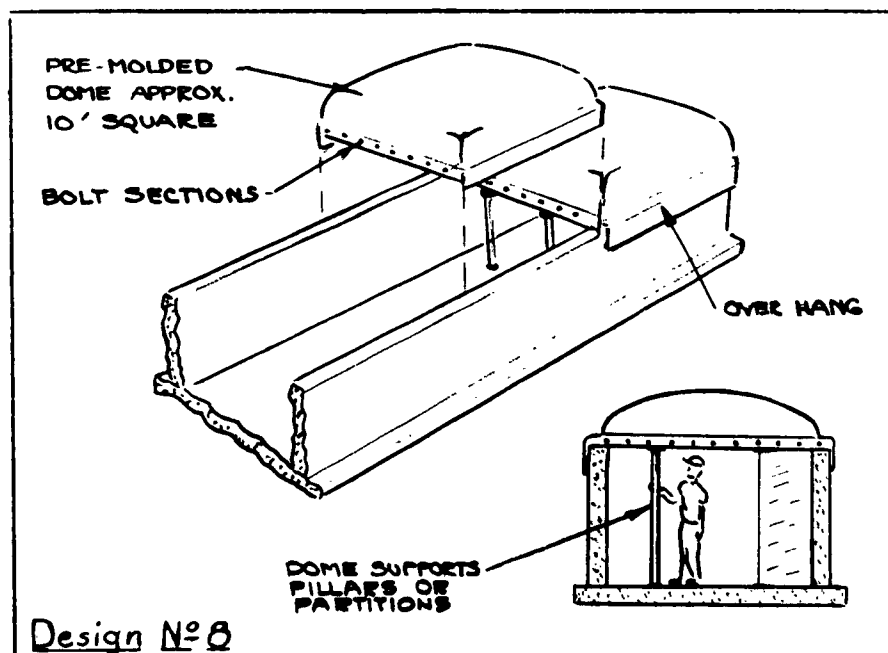


Fig. 34. Roof formed of domes molded from reinforced plastic.
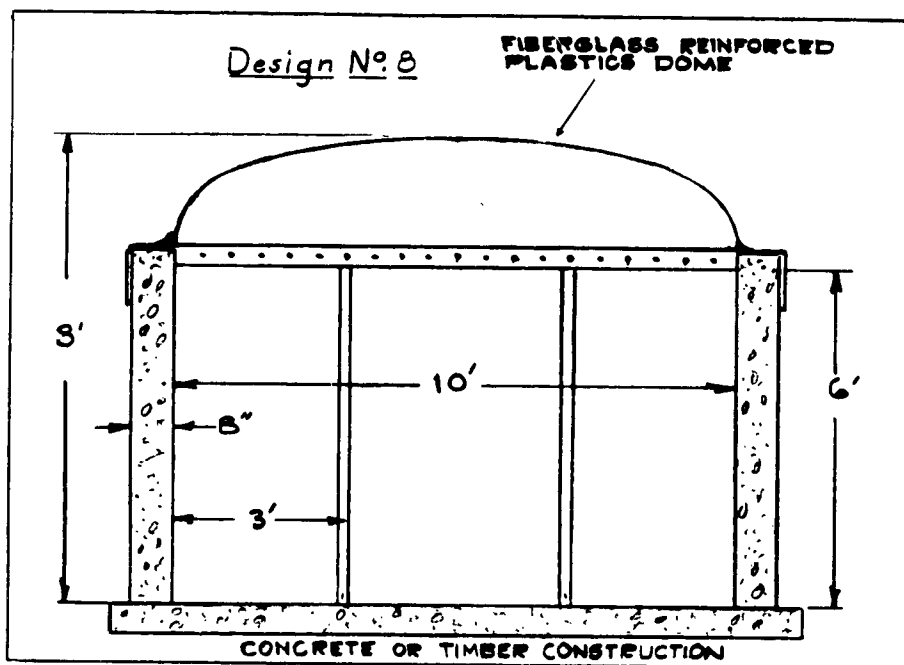
Fig. 35. Longitudinal cross section of roof structure shown in Fig. 34.

The domes would be joined using internal flanges, bolts, and resilient gasket. Either concrete or a reinforced plastic dome section could be employed as the ends of such a structure.

This shelter design would utilize the weight, strength, and rigidity of reinforced concrete to resist floating, together with the lightness, resiliency, and strength of the reinforced plastic domes as resilient and readily portable roof supports.

In situations such as proximity to prime targets, where protection from high overpressures is considered essential, reinforced concrete can again be used.

After the reinforced plastic shelter has been installed in a trench dug no wider than necessary and the backfilling has been completed, a large slab of relatively thick reinforced concrete can be cast over the underground structure at the surface level, with the slab extending out beyond the outline of the shelter underneath by 4 or 5 feet in both directions. This procedure will considerably increase the protection offered to blast overpressures of large magnitude.

The uses of concrete that have been described are refinements or variations that go beyond the basic scope of a structure that can be quickly assembled by inexperienced personnel without the use of special equipment. Based on this requirement, the procedures that would be used in assembling a typical completely enclosed reinforced plastic shelter are considered.

14. Procedure for Installing Reinforced Plastic Shelter Underground. Design 1 is used as an example. Some type of earth moving equipment would be required to make the initial trench. It is felt that a backhoe would be ideal for this purpose as it will dig a trench to the desired depth with relatively vertical walls. This means that a minimum amount of earth is removed, necessitating minimum replacement at the shelter installation.

After the trench has been dug, the floor should be levelled with hand tools and given a loose covering of gravel about 2 inches thick to provide drainage. The cost of gravel varies considerably in different portions of this country. Where it is expensive, some other material such as oyster shells or slag could be substituted.

The bottom shelter sections would then be lowered into place over the gravel bed using ropes restrained by three men on each side of the trench. An "A" frame improvised from timbers would be advantageous for this purpose.

The section flanges would then be fitted with a rubber gasket, "buttered" with a viscous sealant, and bolted together employing rust-resistant steel bolts. Since the sections will weigh under 800 pounds, they can be readily moved horizontally in the trench using pry bars.

If the bottom joining flange is on the outside of the shelter, the floor and end sections would have to be propped up far enough for a man to crawl underneath and bolt the flanges together. In the detailed design of a shelter, every effort should be made to have the flanges on the inside or provide some convenient means of performing this bottom assembly from the inside of the shelter.

After the bottom sections have been assembled, the two center top sections would be lowered and assembled in the same manner using gaskets. After these are in place, the end sections would be lowered and installed.

The entryway, consisting of two sections of reinforced plastic, would then be assembled in the same manner and lowered into place for bolting to one end of the shelter structure.

As any leakage of ground water into the structure is very undesirable, it is important that the structure be tested for such leakage before backfilling is begun.

One very simple and convenient method, employing standby equipment that is available in every community, would be to use the fire department and their pumper. The entire shelter could be quickly filled with water, location of leaks marked, and then the shelter pumped dry using the same piece of equipment.

After testing, the entire exposed portion of the shelter should be wrapped in a layer of thin polyethylene for added moisture protection. This material is available at all building supply outlets, and its cost would be insignificant for a single shelter.

The next operation would be to assemble accessories such as entry ladder and hatch, air inlet and outlet pumps, etc.

The shelter would then be ready for backfilling. This could be done either by hand or with small earth moving equipment such as a tractor or jeep equipped with a bulldozer blade. It is important that a bulldozer is not run over the shelter during the backfilling operation due to possible damage to the shelter and the protruding air vents.

In summation, the largest and heaviest of the component sections are small enough and light enough so that they can be readily handled by a group of six inexperienced men using only a minimum number of hand tools such as timber "A" frames, ropes, and pry bars. Only wrenches would be needed for actual assembly. Using these simple tools, it would be well within the capabilities of amateurs to erect these designs without difficulty.

Table XVI gives estimated assembly times and other pertinent information for the erection of the various shelter designs.

Table XVI. Erection Information

| Design | Minimum Number of Men | Total Man-hours | Elapsed Time (days) | Largest Component | |
|---|---|---|---|---|---|
| | | | | Dimensions (ft) | Weight (lb) |
| 1 - 4 | 6 | 48 | 2 | 9x9x6 | 800 |
| 5 | 6 | 48 | 7 | 9x9x6 | 800 |
| 6 | 4 | 48 | 2 | 2x8x4 | 500 |
| 7 | 4 | 48 | 2 | 10 dia.x6 | 600 |
| 8 | 4 | 16 | 7 | 10x12x2-1/2 | 400 |
| 9 | 6 | 120 | 15 | 12x9x2 | 1000 |

Referring to Table XVI, the second vertical column from the left gives the minimum number of men required to handle the largest and heaviest component. The third column, "Total Man-Hours," represents the total erection time for the shelter proper but does not cover excavation, installation of accessories such as air vents and blowers, or backfilling of the earth around the shelter. The testing of the shelter for leakage is also not included in these estimates.

Erection time estimate for design 5 does not include installation of steel reinforcing rods or the casting of concrete. Similarly, the man-hour assembly time for design 8 does not include the casting of concrete walls and floor.

The fourth column from the left gives the minimum number of elapsed days that would be required for erecting the shelter and backfilling but does not include the excavation time since this might vary considerably. The 7-day figures in this column for designs 5 and 8 include the time required for the concrete to set sufficiently before backfilling is begun. The 15-day figure for design 9 includes the casting of the arches employed in this design. These arches exceed the desired weight limitation of 800 pounds and for this reason would require an "A" frame and a block and tackle for erection.

The last two columns on the right represent the overall dimensions and weight of the largest single component of the plastic portion of this shelter. These columns do not include the weight or dimensions of other components included in the design such as steel reinforcing rods or concrete castings made during installation.

15. Discussion. A detailed study has been made of the requirements of a structural material for use in a community fallout shelter. As a result of this study, glass-reinforced thermosetting plastics have been chosen as a material combination with unusual advantages for this application.

Two plastics, polyester and epoxy resins, have been singled out for use in the binder phase of the combination. For most of the designs considered for such a shelter, polyester resins would be an undisputed choice due to their low price. Epoxies, despite the greater strength that they impart, are sufficiently high in price so that their use can be justified only when the manufacturing process permits them to be used in relatively small proportions with the fibrous reinforcement.

Glass filaments are an uncontested choice as reinforcing material due to the high strength and stiffness that they impart to the plastic. They are low in price and readily available in large quantities.

Two production processes using this material combination are treated in detail in this appendix. The first employs a "spray-up gun" that chops glass roving to short lengths and sprays it with the liquid resin onto a mold surface. This presently produces a relatively low-cost product with a relatively high strength-weight ratio. The same general type of construction and constructions utilizing woven glass fabrics can be laid up by hand.

The second process, roving winding, employs a solid form on which is wound multiple strands of glass which have been wetted with one of the abovementioned plastics in the uncured liquid state. After curing, the epoxy-form of this combination produces the highest strength-weight ratio of any material known today. As this process is in its infancy, the cost of parts made using it is still relatively high and the present manufacturing facilities are limited. However, it is very probable that this cost could be considerably reduced if production facilities were set up for a sufficient volume of production to justify maximum mechanization.

Of the two processes, quantity production of shelters using the "spray-up gun" could be initiated most rapidly. In about 165 manufacturing plants distributed over most of this country (Appendix H), there are presently over 750 spray-up depositor guns which are capable of applying about 10 million pounds of reinforced plastic per year. As these guns are comparatively simple and readily made, additional manufacturing equipment would be no problem. If any urgency is attached to this procurement, the existence of these 165 plants presently equipped to manufacture items similar to shelters from reinforced plastic could be considered a valuable national asset.

An idea has been advanced in this appendix for developing a low-cost, lightweight casting compound that could be employed by inexperienced personnel at the installation location, employing simple forms for the preparation of structural shelter components. These would be cast in much the same way as concrete is presently employed, but the weight of the resulting sections would be considerably less. The potentialities of this approach could be rather readily determined in a development project of limited scope.

An important requirement of this project was the necessity of permitting the installation of the shelter using a limited number

of unskilled men without access to special equipment such as cranes. For all of the designs considered in this appendix, the shelter would be assembled from component parts which are small and light enough to be handled by six men. It is felt that this necessity of assembling a structure of component parts using inexperienced personnel is in itself a serious problem due to the likelihood of ground water seepage into the structure prior to emergency use. It is strongly felt that in any shelter design and evaluation, particular attention should be given to this problem. Ground water leakage will be an important hazard regardless of the materials used in the shelter components.

In this study, sound engineering considerations have been used in evaluating the feasibility of reinforced plastics while the importance of economic factors was also considered. These factors have naturally influenced the consideration and choice of material and manufacturing processes. As a part of this consideration, a brief comparison was made of estimated manufactured cost between structures of identical size and shape made from both steel and reinforced plastic. Steel was chosen as it is felt to present the most serious material competition from a price standpoint.

The estimates show a small monetary advantage for the steel structure as manufactured. However, since the steel shelter would weigh about 3 times as much as one made from plastic, this economic advantage is overcome when the two are each shipped a total of about 700 miles. At that point, the total of manufacture plus shipping cost is equal for the two; beyond this shipping distance, the plastic structure has a definite economic advantage.

Two other factors are also strongly in favor of the reinforced plastic shelter. Unlike the latter which is a good heat insulator, the steel shelter would require an external coating of heat insulating material to provide livable conditions underground. This would add further to its cost.

Secondly, since the steel shelter would weigh three times as much as the equivalent reinforced plastic structure, it would have to be made with three times the number of separate component sections for assembly in order to permit installation without the use of special equipment for handling. This would considerably increase its cost, the time and cost of installation, and the probability of ground water leakage.

Reinforced plastics present the following impressive advantages for use in underground shelters:

a.  The highest strength-weight ratio available from any materials.  (This means that component sections will be lightweight when made from this material combination.  This will reduce shipping costs and simplify installation considerably.)

b.  Resistance to ground water attack and corrosion.

c.  Ease of forming into complex shapes with a minimum tooling investment.

d.  Low thermal conductivity.

This property will promote a more livable interior air temperature in a buried shelter during the winter than either a metal or concrete structure and will eliminate the annoying problem of moisture condensing on the roof and side walls.  However, a reinforced plastic shelter, due to its heat insulating properties, may be difficult to keep comfortably cool on a hot summer day.

After all of the factors involved in the material choice are considered, it is felt that reinforced plastics offer sufficient advantages over all other materials to make them a logical and leading contender for this application.  In a limited period of competing with other materials for the buried home shelter application, for instance, reinforced plastics have already become prominent for their advantages.

The problems connected with the success of underground community shelters are unique and must not be underestimated.  However, they do not present any obstacles that cannot be overcome through a logical process of engineering development.  It is felt that reinforced plastics as structural materials offer unusual promise in the solution of these problems at a minimum overall cost.

# REFERENCES

1. Blackett, Denis A., Feasibility Study Concerning the Utilization of Plastics for Underground Personnel Shelters, sponsored by the U. S. Naval Civil Engineering Research and Evaluation Laboratory, Port Hueneme, California (Massachusetts Institute of Technology, Cambridge, Massachusetts: September 1959).

2. Gibbs & Cox, Inc., Marine Design Manual for Fiberglass Reinforced Plastics (New York, McGraw-Hill Book Company, Inc.), pp 5-1 to 5-44.

3. Hupp, A. G., President, Hupp Systems, Inc., Sarasota, Florida, Personal correspondence to J. R. Fisher (14 April 1962).

4. Fagan, C. D., President, Sharpsville Steel Fabricators, Sharpsville, Pa., Telephone communication to J. R. Fisher (18 April 1962).

5. Modern Plastics Encyclopedia (Bristol, Conn.: Modern Plastics, Inc., 1960 and 1962).

6. Society of the Plastics Industry, Inc., Plastics Engineering Handbook (New York: Reinhold Publishing Corporation, 1954).

7. Sonneborn, Ralph H., Fiberglas Reinforced Plastics (New York, N. Y.: Reinhold Publishing Corp., 1954) pp 175-223.

8. Society of the Plastics Industry, Inc., Plastics in Civil Defense (New York: 1962), pp 19-26.

9. Office of Civil and Defense Mobilization, Technical Report TR-11, "Recommended OCDM Specifications for Blast-Resistant Structural Design" (Washington, D. C.: January 1958).

10. Gurdo, Anthony F., et al, "An Air Force Approach to Rigid Reinforced Plastic Shelters," Report given at 15th Annual Meeting of Reinforced Plastics Division, Society of the Plastics Industry (New York, N. Y.: 1959).

11. Dietz, Albert G. H., "Composite Structures Incorporating Reinforced Plastics," Report given at 17th Annual Meeting of Reinforced Plastics Division, Society of the Plastics Industry (New York, N. Y.: 1961).

176

12. Nuclear Attack and Industrial Survival, special report (New
    York, N. Y.: McGraw-Hill Publishing Co., 1962).

13. "Progress in Spray-Up Means New Markets for Fiber Glass Rein-
    forced Plastics," Modern Plastics, September 1961.

14. "Filament Winding Goes Commercial," Modern Plastics, October
    1961, pp 94-96 and 194.

# APPENDIX G

## DESIGN COMPUTATIONS

Exhibit 1

## TIMBER STRUCTURES

### Terminology*

M   = bending moment

V   = vertical end shear, lb

V'  = modified end shear, lb

$V_2$ = shear at middle support, lb

L   = length of member

f   = unit stress in bending, psi

H   = unit stress in horiz. shear, psi

w   = uniform load per unit length

W   = total uniform load

c   = compressive stress parallel to grain

$c_\perp$ = compressive stress perpendicular to grain

A   = unit area, in.$^2$

d   = width of least side of column, in.

R   = end reaction, lb

$R_2$ = middle reaction, lb

h   = beam depth or thickness, in.

b   = beam width, in.

E   = modulus of elasticity, psi

I   = moment of inertia, in.$^4$

$\Delta$ = deflection, in.

S   = section modulus, in.$^3$

P   = concentrated load, lb

fbm = foot board measure

mbm = thousand board measure

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* As applied to timber

## MAIN SHELTER

10 ft clear span, 5 ft of soil cover.  Weight of soil = 100 lb/ft$^3$

Static load = 5 x 100 x 1/144 = 3-1/2 psi.

$\dfrac{\text{Live Load}}{\text{Total Load}} = \dfrac{20 \text{ psi}}{23\text{-}1/2 \text{ psi}} = 3390 \text{ psf}$

Roof Stringers:

$$M = \frac{wL^2}{8} = \frac{3390}{8} (11)^2 = 51{,}250 \text{ ft-lb}$$

Shear:

$$R \text{ or } V = \frac{3390 \times 11}{2} = 18{,}650 \text{ lb}$$

$$V_{shear} = \text{Ignore}$$

$$H_{shear} = \frac{3V}{2bh} = \frac{3 \times 18{,}650}{2 \times 12 \times h} = \frac{2330}{h}$$

$$h = \frac{2330}{640} = 3.64 = 4'' \text{ min.} \qquad\qquad H = 640 \text{ psi}$$

Bending:

$$M = fS = f \frac{bh^2}{6} \text{ , } \frac{bh^2}{6} = \frac{M}{f} = \frac{51{,}250 \times 12}{6000} = 102.5 \text{ in.}^3$$

$$h^2 = \frac{102.5 \times 6}{12} = 51.25 \text{ in.}^2$$

$$h = 7.17'' = 8'' \text{ min. finished}$$

Compression $\perp$ grain:  Assume 10" cap width, $\dfrac{b}{2} = \dfrac{9\text{-}1/2}{2} = 4\text{-}3/4$ in.

$$c_\perp = \frac{18{,}650}{4\text{-}3/4 \times 12} = \frac{18{,}650}{114} = 328 \text{ psi}$$

Say 8" wide x 8" deep x 11' long = 59 fbm and 151 lb

Caps:

$$w = 23\text{-}1/2 \times 5\text{-}1/2 \times 12^2 = 18{,}650 \text{ lb/ft} \qquad\qquad L = 5'$$

$$M = \frac{wL^2}{8} = \frac{18{,}650 \times (5)^2}{8} = 58{,}250 \text{ ft-lb}$$

Shear:

$$V = \frac{18{,}650}{2} \times 5 = 46{,}600 \text{ lb} \qquad\qquad \text{Assume } b = 10''$$

$$h = \frac{3V}{2Hb} = \frac{3 \times 46{,}600}{2 \times 640 \times 9\text{-}1/2} = 11.50 = 12'' \text{ min.}$$

Bending:

$$M = \frac{fI}{c} = f\,\frac{bh^2}{6}$$

$$\frac{bh^2}{6} = \frac{M}{f} = \frac{58{,}250 \times 12}{6000} = 116.5 \text{ in.}^3$$

$$h^2 = \frac{116.5}{9\text{-}1/2} \times 6 = 73.7 \text{ in.}^2$$

$$h = 8.58 = 10'' \text{ min.}$$

Say 10" wide x 12" deep x 5' long = 50 fbm and 133 lb

Try shear again, ignore loads within beam depth of supports (Reference 2, Appendix B, para. 400-D-2)

$$V' = \frac{W}{2}\left(1 - \frac{2h}{L}\right) = 46{,}600 \;\left(1 - \frac{2 \times 9\text{-}1/2}{5 \times 11\text{-}1/2}\right) = 46{,}600 \;\left(1 - \frac{19}{57\text{-}1/2}\right)$$

$$V' = \frac{W}{2}(1 - 0.330) = \frac{W}{2}(0.67) = 46{,}600\,(0.67) = 31{,}200 \text{ lb}$$

$$h = \frac{3V'}{2Hb} = \frac{3 \times 31{,}200}{2 \times 640 \times 9\text{-}1/2} = 7.70 = 9'' \text{ min.}$$

Use bending depth = 10"

Say 10" wide x 10" deep x 5' long = 42 fbm and 110 lb

Compression $\perp$ grain:  Assume 12" post width

$$c_{\perp} = \frac{46,600}{9\text{-}1/2 \text{ x } 5\text{-}3/4} = 853 < 1100$$

Posts:

W = 18,650 x 5 = 93,250 lb                    h = 10"

$$c = \frac{W}{bh} \; ; \quad 4800 = \frac{93,250}{9\text{-}1/2 \text{ x } b}$$

b = 2.05" = 2-1/2" min.

Say 10" wide x 12" deep x 7' = 70 fbm and 186 lb

Side Loading:

Cohesionless soil   $p = w \, h \, \tan^2 \left(45^0 - \frac{\phi}{2}\right)$   (Reference 10, App. B, p. 50)

where p = static side pressure in psi, w = unit weight of soil, h = depth of soil, $\phi$ = angle of repose of soil.

$$P_{13.5} = \frac{100}{144} \text{ x } 13\text{-}1/2 \tan^2 \left(45^0 - \frac{26}{2}\right) = \frac{1350}{144} \tan^2 (32^0)$$

$$P_{13.5} = 9.37 \text{ x } (0.6249)^2 = 3.66 \text{ psi}$$

$$p_5 = 3.66 \text{ x } \frac{5}{13.5} = 1.355 \text{ psi}$$

Cohesive soil   $p = w \, h \, \tan^2 \left(45^0 - \frac{\phi}{2}\right) - 2c \tan \left(45^0 - \frac{\phi}{2}\right)$
(Reference 10, App. B, p. 60)

where c = cohesive strength of the soil = 400 psf

$$P_{13.5} = \frac{100}{144} \text{ x } 13\text{-}1/2 \tan^2 \left(45^0 - \frac{14}{2}\right) - 2 \text{ x } \frac{400}{144} \tan \left(45^0 - \frac{14}{2}\right)$$

$$P_{13.5} = \frac{1350}{144} \tan^2 (38^0) - \frac{800}{144} \tan (38^0)$$

$$p_{13.5} = 9.37 \ (0.7813)^2 - 5.56 \ (0.7813) = 5.72 - 4.35 = 1.37 \text{ psi}$$

$$p_5 = 5.72 \times \frac{5}{13.5} - 4.35 = 0$$

Assume static side loading = 3-1/2 psi

Assume dynamic side loading = $\frac{20}{2}$ = 10 psi (Reference 3, App. B, p. 55)

Total uniform loading = 13.5 psi
Total uniform loading = 1945 psf

## Posts:

10" x 12" x 7' size at 5' spacing

Uniform load/ft = 5 x 1945 = 9725 lb/ft
Axial load = 5 x 18,650 = 93,250 lb
Span = 7 ft = L, depth = 10" = h, width = 12" = b

$$\frac{P/A}{c} + \frac{M/S}{f} \leq 1$$

$$S = \frac{bh^2}{6} = \frac{11.5 \ (9.5)^2}{6} = 173 \text{ in.}^3$$

$$M = \frac{wL^2}{8} = \frac{9725 \ (7 \times 12)^2}{12 \times 8} = \frac{9725 \ (84)^2}{96} = 715,000 \text{ in.-lb}$$

$$P/A = \frac{93,250}{9.5 \times 11.5} = 854 \text{ psi}$$

$$\frac{P/A}{c} + \frac{M/S}{f} = \frac{854}{4,800} + \frac{715,000}{173 \times 6000} = 0.178 + 0.689 = 0.867 < 1$$

## Side Sheathing:

$$L = 5', \ w = 1945 \text{ lb/ft}, \ b = 12", \ V = \frac{wL}{2} \ , \ V = \frac{1945 \times 5}{2} = 4860 \text{ lb}$$

$$h = \frac{3V}{2bH} = \frac{3 \times 6080}{2 \times 12 \times 640} = 1.19 \qquad \text{say 2" min.}$$

Bending:

$$M = \frac{wL^2}{8} = \frac{1945 \ (5 \times 12)^2}{8 \times 12} = 73,000 \text{ in.-lb}$$

$$h^2 = \frac{6M}{bf} = \frac{6 \times 73,000}{12 \times 6000} = 6.08 \text{ in.}^2$$

$$h = 2.47" \qquad \text{use 3" min.}$$

say 10" x 3" x 5' long = 13 fbm and 31 lb

Scab:

Side loading per longitudinal ft of structure = 13.5 x 8-1/2 x 144 = 16,500 lb

Loading per scab and spreader = $\frac{16,500}{2}$ x 5 = 41,250 lb

Assume uniform loading over total length of scab by drift pin transfer.
Length = twice cap depth.

$$b = 12" \quad L = 20" \quad P = 41,250 \quad w = \frac{41,250}{20} = 2062 \text{ lb/in.}$$

Assume only 1/2 length in bending

$$M = \frac{wL^2}{2} = \frac{2062 \ (10)^2}{2} = 103,100 \text{ in.-lb}$$

$$h^2 = \frac{6M}{bf} = \frac{6 \times 103,100}{11.5 \times 6000} = 8.96"$$

$$h = 2.95 \qquad \text{Say 4", check shear}$$

$$V = \frac{wL}{2} = \frac{2062 \times 10}{2} = 10,320$$

$$h = \frac{3V}{2bH} = \frac{3 \times 10,320}{2 \times 11.5 \times 640} = 2.11 \qquad \text{Say 3", check } c_\perp \text{ due to spreader}$$

$$c_\perp = \frac{P}{A} = \frac{41,250}{5.5 \times 5.5} = 1365 > 1100 \qquad \text{Accept anyway as some compressive failure is allowable}$$

Scab is 12" x 4" x 20" = 7 fbm and 17 lb

Spreaders:

$$A = \frac{P}{c} = \frac{41,250}{4800} = 8.60 \text{ min.} \qquad L = 10' - 2(3.62") = 9.40'$$

$$\text{Assume } d = 5\text{-}1/2 \qquad L/d = \frac{12 \times 9.1}{5.5} = 20.5 < 50$$

$$P/A = \frac{0.30E}{(L/d)^2}, \quad A = \frac{P(L/d)^2}{0.30E} = \frac{41.250 (20.5)^2}{0.30 \times 1,760,000} = \frac{137,500 (20.5)^2}{1,760,000}$$

$$A = 32.8 \text{ in.}^2 \text{ required} \qquad \begin{array}{l} 6" \times 6" = 30.3 \\ 6" \times 8" = 41.25 \end{array}$$

Use 6" x 6" since 6" x 7" is not normally available and 6" x 8" is excessive.

6" x 6" x 9'-4-3/4" = 28 fbm and 70 lb

## Footings for Posts (Sills):

Axial load for posts = 93,250 lb = 93.25$^k$
However, design for dead load only which is 3.5/23.5 x 93.25 = 13.9$^k$

Post load = 3-1/2 x 144 x $\frac{W}{2}$ x L = 3-1/2 x 144 x 5-1/2 x 5

Post load = 13.9$^k$

Allowable soil bearings (from building codes) vary from 2 k/ft$^2$ (soft clay, fine loose sand) to 12 k/ft$^2$ (hard clay).

Based on the minimum above (2 k/ft$^2$), it would be necessary to have a bearing area per post of 7 ft$^2$. With post dimensions of 10" wide and 12" long, there is only 0.83 ft$^2$ available. Using total length for footing, 5 ft between posts, and 10" width, 4 ft$^2$ per post is available. This is one-half the minimum needed. Considering the variation in soil type, this 1/2 figure seems a reasonable compromise. Note: footing area is designed for dead load but footing strength must be designed for dead and live load.

For live load also, use

> 10" x 10" x 5', i. e., same as cap
> Sill and cap are identical

<u>End Bulkhead</u>:

Sheathing - clear span = 10', net span = 10'-10"

Shear:

$$w = 1945 \text{ lb/ft} \qquad L = 10.83' \qquad b = 12"$$

$$V = \frac{wL}{2} = \frac{1945 \times 10.83}{2} = 10,520 \text{ lb}$$

$$h = \frac{3V}{2bH} = \frac{3 \times 10,520}{2 \times 12 \times 640} = 2.06" \qquad \text{Say } 2\text{-}1/2" \text{ min.}$$

Bending:

$$M = \frac{wL^2}{8} = \frac{1945 \ (10.83 \times 12)^2}{8 \times 12} = 343,000 \text{ in.-lb}$$

$$h^2 = \frac{6M}{bf} = \frac{6 \times 343,000}{12 \times 6000} = 28.6 \text{ in.}^2$$

$$h = 5.35" \qquad \text{say } 6" \text{ min.}$$

Use 10" x 6" x 11' long = 55 fbm and 140 lb, check $c_\perp$ on beam

$$c_\perp = \frac{1945 \times 1\text{-}1/2}{12 \times 6} = 149 \text{ psi} < 1100$$

<u>End Post</u>:

L = 7 + 2 x 0.83 = 8.67' overall but 7.83' support to support

$$w = 13.5 \times 144 \ (\frac{10 + 0.83}{2}) = 1945 \times 5.42 = 10,520 \text{ lb/ft}$$

$$V = \frac{wL}{2} = \frac{10,520 \times 7.83}{2} = 41,200 \text{ lb}$$

$$h^2 = \frac{3V}{2H} = \frac{3 \times 41,200}{2 \times 640} = 96.6 \text{ in.}^2$$

h = 9.82", when b = 10", h = 11"

$$M = \frac{wL^2}{8} = \frac{10,520 \times (7.83)^2 \times 12}{8} = 967,000 \text{ in.-lb}$$

$$h^3 = \frac{6M}{f} = \frac{6 \times 967,000}{6000} = 967 \text{ in.}^3$$

$$h = 9.87''$$

By ignoring load within beam depth of support, shear does not govern. Assume b = 10"; therefore, h = 11"

Use 10" x 10" x 8'-8" = 72 fbm and 188 lb

This slight shortness of depth requirement is ignored because of weight.

Axial load on cap:

$$P = 13.5 \times 144 \times \frac{11.58}{2} \times \frac{8.58}{2} = 48,300 \text{ lb}$$

$$\frac{P/A}{c} + \frac{M/S}{f} \leq 1, \quad \frac{bh^2}{6} = \frac{10^3}{6} = 143 \text{ in.}^3$$

$$\frac{48,300}{4800 \times 9.5 \times 9.5} + \frac{58,250 \times 12}{6000 \times 143} \leq 1$$

$$0.112 + 0.814 = 0.926 < 1$$

$$c_{\perp} = \frac{48,300}{9.5 \times 9.5} = 535 \text{ psi} < 1100$$

Deflection:

$$I = \frac{bd^3}{12} = \frac{9.5 \times 9.5^3}{12} = 678 \text{ in.}^4$$

$$\Delta = \frac{5 \ wL^4}{384 \ EI} = \frac{5 \times 10,520 \times (7.83 \times 12)^4}{384 \times 12 \times 1,760,000 \times 678}$$

$$\Delta = \frac{52,600 \times (94)^4}{4610 \times 1,760,000 \times 678} = \frac{11.42 \times (8830)^2}{1,760,000 \times 678} = \frac{77,900,000}{1,760,000 \times 59.4}$$

$$\Delta = 0.75''$$

Spacing of beam required to allow 1" deflection requires that end cap project beyond end post 1 inch. This will prevent additional loading of end post.

Stiffener:

Use 2" x 4" as erection stiffener

$\sin \theta = \dfrac{1.625}{d}$

$d = 1.625/\sin \theta$

$\sin \theta \approx \dfrac{48.5}{\sqrt{(81)^2+(48.5)^2}}$

$\sin \theta \approx 0.52$

$d = 1.625/0.52 = 3.13"$

$a = 1.625/\tan \theta = \dfrac{1.625}{0.609}$

$a = 2.67" = 2\text{-}11/16"$
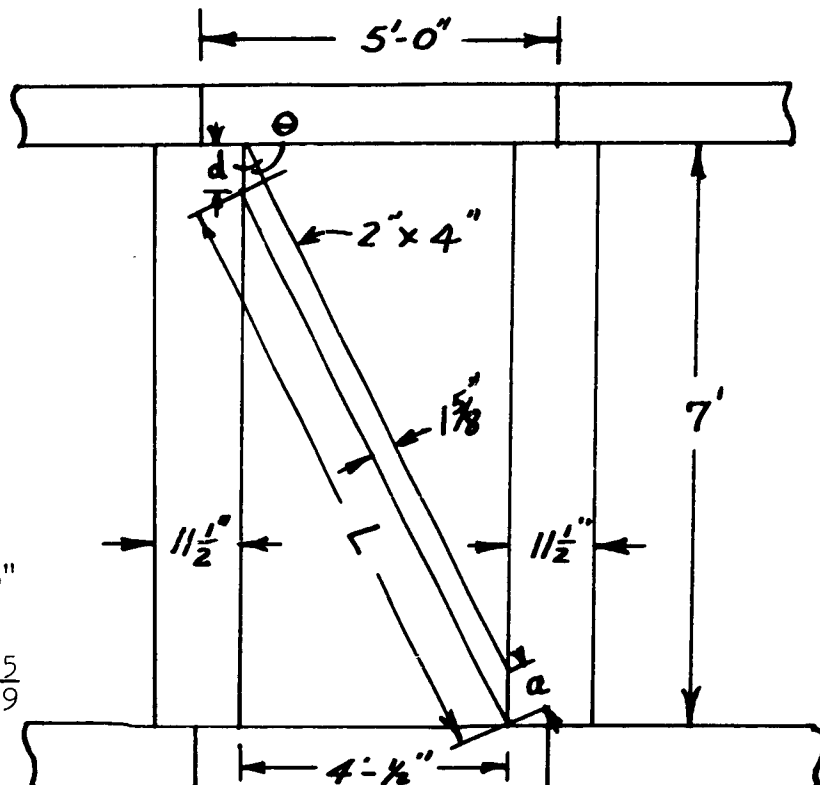
$L = \sqrt{(84 - 3.13)^2 + (48.5)^2} = \sqrt{8850} = 94.3" = 7'\text{-}10\text{-}5/16"$

$L + a = 7'\text{-}10\text{-}5/16" + 2\text{-}11/16" = 8'\text{-}1"$

Use 2" x 4" x 8'-1" = 5.4 fbm and 11.3 lb each

Required - 24

Entrance Bulkhead:

Total load = 1945 (10' + 9-1/2" + 9-1/2")(7' + 9-1/2" + 9-1/2") =

1945 (11.58')(8.58) = 193,500 lb

Gross area = 11.58 x 8.58 = 99.4 ft$^2$

Net area = gross area less entrance area

$$= 99.4 - 7 (3' + 11'' + 5\text{-}1/4'') = 99.4 - 7 (4.35) =$$

$$99.4 - 30.45 = 68.95 \text{ ft}^2$$

Uniform load against net area $= 1945 \dfrac{99.4}{68.95} = 2800 \text{ lb/ft}^2$

**Horizontal End Beams:**    2 required, L = 10' + 9-1/2" = 10.79"



b = 10"
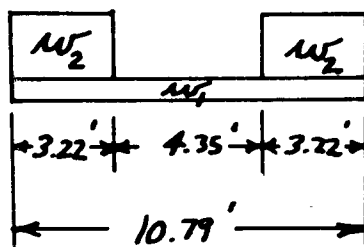
$w_1 = 2800 \times \dfrac{9.5}{12} = 2210 \text{ lb/ft}$

$w_1$ = uniform load against beam itself

$w_2$ = uniform load from vertical stringers

$w_2 = 2800 \times \dfrac{7}{2} = 9800 \text{ lb/ft}$

Shear:

$$V = \dfrac{w_1 L}{2} + 3.22\, w_2 = \dfrac{2210 \times 10.79}{2} + 3.22 \times 9400 = 12{,}000 + 31{,}500$$

$$V = 43{,}500 \text{ lb}$$

$$h = \dfrac{3V}{2bH} = \dfrac{3 \times 43{,}500}{2 \times 9.5 \times 640} = 10.72''$$

Bending:

$$M = \dfrac{w_1 L^2}{8} + 5.18\, w_2 = \dfrac{2210\, (10.79)^2}{8} + 5.18 \times 9800 = 32{,}200 + 50{,}800$$

$$M = 83{,}000 \text{ ft-lb} = 996{,}000 \text{ in.-lb}$$

$$h^2 = \dfrac{6M}{bf} = \dfrac{6 \times 996{,}000}{9.5 \times 6{,}000} = 104.8 \text{ in.}^2$$

$$h = 10.24''$$

The overage on beam depth requirement for shear can be disregarded by ignoring loads within beam depth of supports.

$$V' = \dfrac{wL}{2} \left(1 - \dfrac{2h}{L}\right) = 12{,}000 \left(1 - \dfrac{1.58}{10.79}\right) + 31{,}500 \left(1 - \dfrac{1.58}{3.22}\right)$$

$V' = 12,000 (1 - 0.146) + 31,500 (1 - 0.491) = 12,000 (0.854) +$

$\qquad 31,500 (0.509)$

$V' = 10,240 + 16,040 = 26,280$ lb

$h = \dfrac{3V'}{2bH} = \dfrac{3 \times 26,280}{2 \times 9.5 \times 640} = 6.25"$

Due to large weight of this beam, the overage on beam depth required for bending is ignored.

Use 10" x 10" x 11'-7" = 97 fbm and 253 lb

Axial load on cap = $V_{beam} + (w_1 + w_2) 0.39 = 43,500 + 12,010 (0.39)$

$\qquad\qquad = 43,500 + 4680 = 48,180$ lb which is the same
$\qquad\qquad$ load from bulkhead at other end.

Vertical Sheathing:

Assume b = 12" net, L = 7' + 9-1/2" = 7.79'

$\qquad w = \dfrac{2684 \times 12}{12} = 2684$ lb/ft

Shear:

$\qquad V = \dfrac{wL}{2} = \dfrac{2684 \times 7.79}{2} = 10,440$ lb

$\qquad h = \dfrac{3V}{2bH} = \dfrac{3 \times 10,440}{2 \times 12 \times 610} = 2.04"$

Bending:

$\qquad M = \dfrac{wL^2}{8} = \dfrac{2684 \times (7.79)^2}{8} \times 12 = 244,600$ ft-lb

$\qquad h^2 = \dfrac{6M}{bf} = \dfrac{6 \times 244,600}{12 \times 6000} = 20.4$ in.$^2$

$\qquad h = 4.52"\qquad$ Use 5"

Use 8" x 5" x 8'-6" = 28 fbm and 70 lb

Quantity required = 6 per side = 12

Note: Gap between top of entranceway stringers and lower edge of top horizontal end beam is filled by one 16" x 5" x 5'-8" end sheathing.

## ENTRANCEWAY

Assume 3' wide and 5' high clear. Floor shall be 9-1/2" above shelter floor.

Earth cover is $5' + 10" + 8" = 6\text{-}1/2'$ approximately

Total load = $6\text{-}1/2 \times \frac{100}{144} + 20 = 24\text{-}1/2$ psi = 3530 psf

Roof Stringers:

$$M = \frac{wL^2}{8} = \frac{3530}{8} (3.5)^2 = 5410 \text{ ft-lb} = 64,900 \text{ in.-lb}$$

Shear:

$$V = \frac{3530 \times 3.5}{2} = 6175 \text{ lb} \qquad \text{Assume } b = 12"$$

$$h = \frac{3V}{2bH} = \frac{3 \times 6175}{2(12) \ 640} = 1.2"$$

Bending:

$$h^2 = \frac{6M}{bf} = \frac{6 \times 64,900}{12 \times 6,000} = 5.41 \text{ in.}^2$$

$$h = 2.32" \qquad \text{Say } 3", \quad L = 3'\text{-}11"$$

$$c_{\perp} = \frac{6175}{6 \times 12} = \frac{6175}{72} = 86 \text{ psi} \qquad \text{Assumed cap width = 6"}$$

Short Caps: 4 required (2 as sills)

$$w = 3530 \times 4/2 = 7060 \text{ lb/ft}$$

Shear:

$$V = \frac{wL}{2} = \frac{7060 \times 5}{2} = 17,650 \text{ lb}, \qquad L = 5'$$

$$h^2 = \frac{3V}{2H} = \frac{3 \times 17,650}{2 \times 640} = 41.4 \text{ in.}^2$$

$$h = 6.43"$$

192

Bending:

$$M = \frac{wL^2}{8} = \frac{7060 \times (5)^2}{8} = 22{,}050 \text{ ft-lb}$$

$$h^3 = \frac{6M}{f} = \frac{6 \times 22{,}050 \times 12}{6000} = 265 \text{ in.}^3$$

$h = 6.42"$     use $b = 6"$, $h = 8"$, ignore shear loads within cap
depth from post

Say 6" x 8" x 5'-9-1/2" = 23 fbm and 58 lb

Long Caps:  4 required (2 as sills)

L = 4' assumed, two span continuous

Shear:

$$V = \frac{5wL}{8} = \frac{5 \times 7060 \times 4}{8} = 17{,}650 \text{ lb/ft, } b = 6"$$

$$h = \frac{3V}{2Hb} = \frac{3 \times 17{,}650}{2 \times 640 \times 5.5}$$

$h = 7.52"$, $b = 6"$, $h = 8"$

Bending:

$$M = \frac{wL^2}{8} = \frac{7060 \times (4)^2}{8} = 14{,}120 \text{ ft-lb}$$

$$h^2 = \frac{6M}{bf} = \frac{6 \times 14{,}120 \times 12}{5.5 \times 6000} = 30.8 \text{ in.}^2$$

$h = 5.56"$

Say 6" x 8" x 7'-8-1/2" = 31 fbm and 77 lb

Side loading:

Cohesionless soil   $p = wh \tan^2 (45° - \frac{\phi}{2})$

$$p_{12} = \frac{100}{144} \times 12 \tan^2 (45° - \frac{26}{2}) = \frac{1200}{144} \tan^2 (32°)$$

$$p_{12} = 8.33 \times (0.6249)^2 = 3.26 \text{ psi}$$

$$P_{6.5} = 3.26 \times \frac{6.5}{12} = 1.764 \text{ psi}$$

Assume static side loading = 3 psi
Assume dynamic side loading = 10 psi = $\frac{20}{2}$ psi
Total uniform side loading = 13 psi
Total uniform side loading = 1872 psf

## Posts for Short Caps: 4 required

$$W = 7060 \times 5 = 35,300 \text{ lb} \qquad h = 6"$$

Compression:

$$b = \frac{W/2}{hc} = \frac{17,650}{5.5 \times 4800} = 0.67" \text{ min.}$$

Compression $\perp$ grain for cap: $\qquad h = 6"$

$$b = \frac{V}{hc_\perp} = \frac{17,650}{55 \times 1100} = 2.92" \text{ min.}$$

Side loading:

Uniform load/ft = $\frac{5.67}{2} \times 1872 = 5300$ lb/ft

Axial load = $\frac{5.67}{2} \times 7060 = 20,000$ lb

Span = 5 ft = L, depth = 6" = h, width = 6" = b

$$\frac{P/A}{c} + \frac{M/S}{f} \leq 1, \quad \frac{P}{Ac} + \frac{M}{Sf} \leq 1$$

$$S = \frac{bh^2}{6} = \frac{5.5 (5.5)^2}{6} = 27.7 \text{ in.}^3$$

$$M = \frac{wL^2}{8} = \frac{5300 (5 \times 12)^2}{12 \times 8} = 199,000 \text{ in.-lb}$$

$$A = bh = 5.5 \times 5.5 = 30.3 \text{ in.}^2$$

$$\frac{20,000}{30.3 \times 4800} + \frac{199,000}{27.7 \times 6000} = 0.138 + 1.200 > 1$$

**Try b = 8"** $\qquad$ A = 5.5 × 7.5 = 41.25 in.$^2$

$$S = \frac{bh^2}{6} = 7.5 \frac{(5.5)^2}{6} = 37.8 \text{ in.}^3$$

$$\frac{20,000}{41.25 \times 4800} + \frac{199,000}{37.8 \times 6000} = 0.101 + 0.878 < 1$$

Use 8" x 6" x 5' = 20 fbm and 57 lb

<u>Middle Post for Long Caps, Long Side:</u> 1 required

L = 4', h = 6"

$$V_2 = \frac{7060 \times 4 \times 5}{8} = 17,650 \text{ lb}$$

Compression:

$$b = \frac{2V_2}{hc} = \frac{35,300}{5.5 \times 4800} = 1.34" \text{ min.}$$

Compression $\perp$ grain for cap:      h = 6"

$$b = \frac{2V_2}{hc_\perp} = \frac{2 \times 17,650}{5.5 \times 1100} = 5.84" \qquad 8" \text{ x } 6" \text{ should be suitable}$$

Side loading:

Uniform load = $\frac{1}{2}$ x 4 x 1872 x 2 = 7488 lb/ft

Axial load = 2 $V_2$ = 35,300 lb

Span = 5 ft = L;    depth = 6" = h;    width = 8" = b

$$\frac{P/A}{c} + \frac{M/S}{f} \leq 1 \quad \frac{P}{Ac} + \frac{M}{Sf} \leq 1$$

$$S = \frac{bh^2}{6} = 7.5 (5.5)^2 = 37.8 \text{ in.}^3, \ A = 5.5 \times 7.5 = 41.25 \text{ in.}^2$$

$$M = \frac{wL^2}{8} = \frac{7488 \times (5 \times 12)^2}{12 \times 8} = 280,500 \text{ in.-lb}$$

$$\frac{35,000}{41.25 \times 4800} + \frac{280,500}{37.8 \times 6000} = 0.179 + 1.237 > 1$$

Try 12" x 6"

$$S = \frac{11.5 \ (5.5)^2}{6} = 57.9 \text{ in.}^3 \qquad A = 11.5 \times 5.5 = 63.25 \text{ in.}^2$$

$$\frac{35,300}{63.25 \times 4800} + \frac{280,500}{57.9 \times 6000} = 0.116 + 0.808 < 1$$

Use 12" x 6" x 5' = 30 fbm and 77 lb

<u>End Posts for Long Caps, Long Side:</u>  2 required

$$P = 3/5 \ V_2 = \frac{3 \times 17,650}{5} = 10,600 \text{ lb}$$

End loading = $1872 \times \frac{4}{2} = 3744$ lb/ft of post

Side loading = $1872 \times \frac{4}{2} = 3740$ lb/ft

Try b = 6", h = 6", A = 30.3 in.$^2$, $S_{se} = \frac{bh^2}{6} = \frac{(5.5)^3}{6} = 27.7$ in.$^3$

$$\frac{P}{Ac} + \frac{M_s}{S_e f} + \frac{M_e}{S_s f} \leq 1 \qquad \begin{array}{l} \text{where } M_s = \text{side moment} \\ \qquad M_e = \text{end moment} \\ \qquad S \ = S_s = S_e \end{array}$$

$$M_e = \frac{wL^2}{8} = \frac{3744 \times (5 \times 12)^2}{12 \times 8} = 140,100 \text{ in.-lb}$$

$$M_s = \frac{wL^2}{8} = \frac{3744 \times (5 \times 12)^2}{12 \times 8} = 140,100 \text{ in.-lb}$$

$$\frac{10,600}{30.3 \times 4800} + \frac{140,100 + 140,100}{27.7 \times 6000} = 0.073 + 1.69 > 1$$

Try 6" x 10" with long dimension perpendicular to end load

$$S_s = \frac{9.5 \ (5.5)^2}{6} = 47.9 \text{ in.}^3 \qquad S_e = \frac{5.5 \ (9.5)^2}{6} = 82.7 \text{ in.}^3$$

A = 5.5 x 9.5 = 52.25 in.$^2$

$$\frac{10,600}{52.25 \times 4800} + \frac{140,100}{82.7 \times 6000} + \frac{140,100}{47.9 \times 6000} = 0.043 + 0.282 + 0.488 < 1$$

Use 6" x 10" x 5' for double side-loaded post and 6" x 6" x 5'
for single side-loaded post

Middle Post for Long Cap, Short Side:  1 required

$$V_2 = \frac{7060 \times 4 \times 5}{8} = 17,650 \text{ lb} \qquad L = 4'; \quad b = 6"$$

Compression $\perp$ grain for cap:

$$h = \frac{2V_2}{bc_\perp} = \frac{35,300}{5.5 \times 1100} = 5.85" \text{ min.; } \quad \text{however, some failure is allowable}$$

Side loading:

Uniform load $= \dfrac{4 \times 1872}{2} = 3744$ lb/ft

Axial load $= 2V_2 = 35,300$ lb

Span $= 5$ ft $= L$, $b = 6"$, try $h = 6"$

$$S = \frac{bh^2}{6} = \frac{5.5\,(5.5)^2}{6} = 27.7 \text{ in.}^3, \quad A = bh = 5.5 \times 5.5 = 31.3 \text{ in.}^2$$

$$M = \frac{wL^2}{8} = \frac{3744\,(5 \times 12)^2}{12 \times 8} = 141,000 \text{ in.-lb}$$

$$\frac{P/A}{c} + \frac{M/S}{f} \leq 1, \quad \frac{P}{Ac} + \frac{M}{Sf} \leq 1$$

$$\frac{35,300}{31.3 \times 4800} + \frac{141,000}{27.7 \times 6000} = 0.235 + 0.849 > 1$$

Use 10" x 6" x 5' = 25 fbm and 64 lb to provide adequate bearing surface for siding

End Posts for Long Cap, Short Side:  2 required

$$\text{Axial load} = V = \frac{7060 \times 4 \times 3}{8} = 10,600 \text{ lb}$$

Compression $\perp$ grain for cap:

$$h = \frac{V}{bc_\perp} = \frac{10,600}{5.5 \times 1100} = 1.75" \text{ min.}$$

Side loading:

Uniform load $= \dfrac{4 \times 1872}{2} = 3744$ lb/ft for side-loaded post

Uniform load = $\dfrac{4 \times 1872}{2}$ = 3744 lb/ft for end-loaded post

Try 6" x 6"    S = 27.7 in.$^3$, A = 30.3 in.$^2$

$M = \dfrac{wL^2}{8}$ = 141,000 in.-lb

$\dfrac{10{,}600}{30.3 \times 4800} + \dfrac{141{,}000}{27.7 \times 6000}$ = 0.073 + 0.849 < 1 OK for side-
loaded post

Also OK for end-loaded post

Use 6" x 6" x 5' = 15 fbm and 37 lb

## Side Sheathing for Short Cap Section

Single span, L = 5', w = 1872 psf, b = 12"

Shear:

$h = \dfrac{3V}{2bH} = \dfrac{3wL}{4bH} = \dfrac{3 \times 1872 \times 5}{4 \times 12 \times 640}$ = 0.913"

Bending:

$M = \dfrac{wL^2}{8} = \dfrac{1872\ (5 \times 12)^2}{12 \times 8}$ = 70,100 in.-lb

$h^2 = \dfrac{6M}{bf} = \dfrac{6 \times 70{,}000}{12 \times 6{,}000}$ = 5.84 in.$^2$

h = 2.42"     Use 10" x 3" x 5'-7-3/8" short side, 10" x 3" x
9'-8-1/2" long side, 8 required per side.

## Long Side Sheathing for Long Cap Section:

L = 4', w = 1872 psf, b = 12"

Shear:

$h = \dfrac{3V}{2bH} = \dfrac{wL \times 3}{4bH} = \dfrac{3 \times 1872 \times 4}{4 \times 12 \times 640}$ = 0.732"

Bending:

$M = \dfrac{wL^2}{8} = \dfrac{1872 \times (4 \times 12)^2}{12 \times 8}$ = 44,900 in.-lb

$$h^2 = \frac{6M}{bf} = \frac{6 \times 44,900}{12 \times 6000} = 3.75 \text{ in.}^2$$

$h = 1.94"$      Use 10" x 2-1/2" x 7'-8-1/2" = 16 fbm and 38 lb, 8 required

## Short Side Sheathing for Long Cap Section:

Single Span, L = 4', w = 1872 psf, b = 12"

Shear:

$$h = \frac{3V}{2bH} = \frac{3wL}{4bH} = \frac{3 \times 1872 \times 4}{4 \times 12 \times 640} = 0.732"$$

Bending:

$$M = \frac{wL^2}{8} = \frac{1872 \ (4 \times 12)^2}{12 \times 8} = 44,900 \text{ in.-lb}$$

$$h^2 = \frac{6M}{bf} = \frac{6 \times 44,900}{12 \times 6000} = 3.74 \text{ in.}^2$$

$h = 1.93"$      Use 10" x 2-1/2" x 3'-9-1/2" = 8 fbm and 19 lb

## End Sheathing for Long Cap Section:

Single span, L = 3.5', w = 1872 psf, b = 12"

Shear:

$$h = \frac{3V}{2bH} = \frac{3wL}{4bH} = \frac{3 \times 1872 \times 3.5}{4 \times 12 \times 640} = 0.639"$$

Bending:

$$M = \frac{wL^2}{8} = \frac{1872 \ (3.5 \times 12)^2}{12 \times 8} = 34,400 \text{ in.-lb}$$

$$h^2 = \frac{6M}{bf} = \frac{6 \times 34,400}{12 \times 6000} = 2.87 \text{ in.}^2$$

$h = 1.69"$      Use 10" x 3" x 9'-8-1/2" = 8 fbm and 20 lb
                   (same as short cap section, make continuous)

## Scab, Short Cap Section: 4 required

Side loading per longitudinal ft of structure = 13 x 6.5 x 144

$$= 12,180 \text{ lb}$$

Loading per scab and spreader = $\frac{12,180}{2}$ x $\frac{5.5}{2}$ = 16,750 lb

Assume uniform loading over total length of scab by drift pin transfer.

Length = twice cap depth = 2 x 7.5 = 15"

b = 10", P = 16,750, w = $\frac{16,750}{15}$ = 1117 lb/in.

Assume only 1/2 length in bending

$M = \frac{wL^2}{2} = \frac{1117 \ (7.5)^2}{2} = 31,400$ in.-lb

$h^2 = \frac{6M}{bf} = \frac{6 \ x \ 31,400}{9.5 \ x \ 6000} = 3.31$ in.$^2$

h = 1.82" check shear

$V = \frac{wL}{2} = \frac{1117 \ x \ 7.5}{2} = 4,190$ lb

$h = \frac{3V}{2bH} = \frac{3 \ x \ 4190}{2 \ x \ 9.5 \ x \ 640} = 1.04$" Check $c_\perp$ spreader

$c_\perp = \frac{P}{A} = \frac{16,750}{3.62 \ x \ 3.62} = 1280$ psi > 1100    Accept anyway as some failure is tolerable

Use 10" x 2-1/2" x 15" = 3 fbm and 6 lb

Spreaders, Short Cap Section: 4 required

$A = \frac{P}{c} = \frac{16,750}{4800} = 3.49$ in.$^2$, L = 3' - 2 (2-1/8") = 2.65 ft

Assume d = 3-5/8"    $d = \frac{2.65 \ x \ 12}{50} = 0.64$" min.

$L/d = \frac{2.65 \ x \ 12}{3.62} = 8.79$

$P/A = \frac{0.30E}{(L/d)^2}$ , $A = \frac{P(L/d)^2}{0.30E} = \frac{16,750 \ (8.79)^2}{0.30 \ x \ 1,760,000} = 2.45$ in.$^2$

Use 4" x 4" x 2'-7-3/4", A = 3.62 x 3.62 = 13.14 in.$^2$,
L = 2'-7-3/4" = 4 fbm and 9 lb

Excessive size used to reduce $c_\perp$ on scab

## End Scabs, Long Cap Section: 4 required

Side loading = 13 x 6.5 x 144 = 12,180 lb/ft of structure

Loading per scab and spreader = $\frac{12,180}{2}$ x $\frac{4.5}{2}$ = 13,750 lb

Assume uniform loading over total length of scab by drift pin transfer.

Length = twice cap depth = 2 x 7.5 = 15"

$b = 6"$, $P = 13,750$ lb, $w = \frac{13,750}{15}$ = 917 lb/in.

Assume only 1/2 length in bending

$M = \frac{wL^2}{2} = \frac{917\ (7.5)^2}{2} = 25,800$ in.-lb

$h^2 = \frac{6M}{bf} = \frac{6\ x\ 25,800}{5.5\ x\ 6000} = 4.69$ in.$^2$

$h = 2.16"$, check shear, $V = \frac{wL}{2} = \frac{917\ x\ 7.5}{2} = 3440$ lb

$h = \frac{3V}{2bH} = \frac{3\ x\ 3440}{2\ x\ 5.5\ x\ 640} = 1.48"$, check $c_\perp$ due to spreader

$c_\perp = \frac{P}{A} = \frac{13,750}{3.62\ x\ 3.62} = 1044$ psi < 1100

Use 6" x 3" x 15" = 2 fbm and 5 lb each

## Middle Scabs, Long Cap Section: 2 required

Side loading = 13 x 6.5 x 144 = 12,180 lb/ft of structure

Loading per scab and spreader = $\frac{12,180}{2}$ x 4.5 = 27,500 lb

$b = 8"$, $P = 27,500$, $w = \frac{27,500}{15}$ = 1830 lb/in., $L = 2$ x 7.5 = 15"

Assume only 1/2 length in bending

$$M = \frac{wL^2}{2} = \frac{1830 \ (7.5)^2}{2} = 51,500 \text{ in.-lb}$$

$$h^2 = \frac{6M}{bf} = \frac{6 \times 51,500}{7.5 \times 6000} = 6.87 \text{ in.}^2$$

$$h = 2.62", \text{ check shear, } V = \frac{wL}{2} = \frac{1830 \times 7.5}{2} = 6860 \text{ lb}$$

$$h = \frac{3V}{2bH} = \frac{3 \times 6860}{2 \times 7.5 \times 640} = 2.15", \text{ check } c_\perp \text{ due to spreader}$$

$$c_\perp = \frac{P}{A} = \frac{27,500}{4.5 \times 5.5} = 1112 \text{ psi} > 1100 \text{ some failure is allowable}$$

Use 8" x 3" x 15" = 3 fbm and 6 lb

### End Spreaders, Long Cap Section:  4 required

$$A = \frac{P}{c} = \frac{13,750}{4800} = 2.87 \text{ in.}^2, \quad L = 3' - 2(2\text{-}5/8") = 2.56 \text{ ft}$$

$$\text{Assume d} = 2\text{-}5/8" \quad L/d = \frac{2.56 \times 12}{2.62} = 11.8 < 50$$

$$A = \frac{P \ (L/d)^2}{0.30 \ E} = \frac{13,750 \times (11.8)^2}{0.30 \times 1,760,000} = 3.63" \text{ required}$$

Use 4" x 4" x 2' - 6-3/4", A = 3.62 x 3.62 = 13.14 in.$^2$, 4 fbm and 8 lb

Excessive size used to reduce $c_\perp$ on scab

### Middle Spreaders, Long Cap Section:  2 required

$$A = \frac{P}{c} = \frac{27,500}{4800} = 5.73 \text{ in.}^2 \text{ min., } L = 3' - 2(2\text{-}5/8") = 2.56 \text{ ft}$$

$$\text{Assume d} = 4\text{-}1/2" \quad L/d = \frac{2.56 \times 12}{4.50} = 6.72 > 50$$

$$A = \frac{P \ (L/d)^2}{0.30E} = \frac{27,500 \times (6.72)^2}{0.30 \times 1,760,000} = 2.35 \text{ in.}^2 \text{ min.}$$

Use 5" x 6" x 2' - 6-3/4", A = 4.5 x 5.5 = 24.75 in.$^2$, 6 fbm and 15 lb

Spreaders are placed with bottom edge level with cap bottom edge

## Longitudinal Braces, Long Cap Section: 8 required

Axial loading $= \frac{1}{4} \times 13 \times 6.5 \times 144 \times 4 = P = 12{,}180$ lb

$A = \frac{P}{c} = \frac{12{,}180}{4800} = 2.54$ in.$^2$, $L = 3.00$ ft max.

Assume $d = 2\text{-}1/8"$   $L/d = \frac{3.00 \times 12}{2.12} = 17.00 < 50$

$A = \frac{P\,(L/d)^2}{0.30E} = \frac{12{,}180\,(17.00)^2}{0.30 \times 1{,}760{,}000} = 6.66$ in.$^2$

$c_{\perp}$ on post $= \frac{12{,}180}{A}$ ,   $A = \frac{12{,}180}{1100} = 11.07"$ desired min.

$b = 6"$ to keep head space reduction to a minimum

Use $6" \times 2\text{-}1/2" \times 3'$ for four longest braces, $c_{\perp} < 1100$ as
$A = 11.67$ in.$^2$

Remaining four are shorter.

Braces are placed at extreme edges of posts, top and bottom

## Sills:

Total load on sills $= 4 \times 3530 \times 1/2 = 7060$ lb/ft of structure

Dead load on posts $= 7060 \times \frac{4.5}{24.5} = 1300$ lb/ft

Bearing area of sill per foot $= 1 \times \frac{5.5}{12} = 0.458$ ft$^2$

Load bearing capability of soil $= \frac{1300}{0.458} = 2840$ lb/ft$^2$ required

Since bearing area of poorest soil is 2000 lb/ft$^2$, the $6"$ wide
sills are adequate.

Entrance Vertical Hatch:

Dimensions:

3 ft square, height = 5' + 7' + 7-1/2" + 9-1/2" - 1' + 7-1/2"

$$- 1' = 10' + 24\text{-}1/2" = 12'\text{-}1/2"$$

where height measures from base of horizontal sill to 1 ft below surface

Assume corner posts (vertical beam) are supported at ends and middle to form a two-span continuous beam uniformly loaded.

Static side loading = 3.0 psi at 13-ft depth

Dynamic side loading = 10.0 psi = $\frac{20}{2}$

Total side loading = 13 psi = 1872 psf

Siding:

$$L = 3' + 2 \left(\frac{\text{width of post}}{2}\right) = \text{say } 3.67'$$

$$w = \frac{1872}{1} = 1872 \text{ lb/ft}, \quad b = 12"$$

$$M = \frac{wL^2}{8} = \frac{1872 \ (3.67)^2}{8} = 3150 \text{ ft-lb} = 37,800 \text{ in.-lb}$$

Shear:

$$V = \frac{wL}{2} = \frac{1872 \times 3.67}{2} = 3435 \text{ lb}$$

$$h = \frac{3V}{2bH} = \frac{3 \times 3435}{2 \times 12 \times 640} = 0.67"$$

Bending:

$$h^2 = \frac{6M}{bf} = \frac{6 \times 37,800}{12 \times 6000} = 3.15 \text{ in.}^2$$

$$h = 1.78"$$

Use 12" x 2-1/2" x 4'-3" = 11 fbm and 26 lb

<u>Vertical Beam</u>:   $L = \frac{12'}{2} = 6'$, 4 required, two span continuous

Loading per ft of post $= \frac{1872}{1} \times (\frac{3' + 2b}{2}) = 1872 \times \frac{4.33}{2} = 4055$ lb/ft

Shear:

$V_2 = \frac{5wL}{8} = \frac{4055 \times 5 \times 6}{8} = 15,200$ lb, try b = 6"

$h^2 = \frac{3V}{2H} = \frac{3 \times 15,200}{2 \times 640} = 35.70$ in.$^2$

$h = 5.92"$     A 7" x 7" would be suitable

<u>Note</u>:   Beam is loaded in two directions, $90°$ to each other; there-
fore, bending moment is doubled.

$M = 2 \times \frac{wL^2}{8} = \frac{2 \times 4055 \times (6 \times 12)^2}{12 \times 8} = 437,500$ in.-lb

$h^3 = \frac{6M}{f} = \frac{6 \times 437,500}{6000} = 437.5$ in.$^3$

$h = 7.59"$

Use 8" x 8"

8" x 8" x 12' - 2" = 65 fbm and 168 lb

<u>Transverse Braces</u>:   12 required, 4 at each support point

Middle brace;   $P = R_2$     4 required

$R_2 = 2V_2 = \frac{2 \times 5wL}{8} = \frac{10 \times 4055 \ (6)}{8} = 30,400$ lb

Assume unsupported column, L = 3'

Area $= \frac{P}{c} = \frac{30,400}{4800} = 6.34$ in.$^2$

Assume d = 6"    $L/d = \frac{3 \times 12}{5.5} = 6.55 < 50$

$$A = \frac{P \ (L/d)^2}{0.30E} = \frac{30,400 \times (6.55)^2}{0.30 \times 1,760,000} = 2.48 \ in.^2$$

Check compression on beam, $A = \frac{P}{c_{\perp}} = \frac{30,400}{1100} = 27.7 \ in.^2$ required

Use 6" x 6", $A = 30.25 \ in.^2$

End Braces: $P = R_1 = R_3$      8 required

$$R_1 = V_1 = \frac{3wL}{8} = \frac{3 \times 4055 \times (6)}{8} = 9,125 \ lb$$

$$Area = \frac{P}{c} = \frac{9125}{4800} = 1.91 \ in.^2 \qquad L = 3'$$

Assume d = 3"    $L/d = \frac{3 \times 12}{2.62} = 13.8 < 50$

$$A = \frac{P \ (L/d)^2}{0.30E} = \frac{9,125 \times (13.8)^2}{0.30 \times 1,760,000} = 0.33 \ in.^2$$

Check compression on beam, $A = \frac{P}{c_{\perp}} = \frac{9125}{1100} = 8.3 \ in.^2$ required

Use 4" x 4" x 3' = 3 fbm and 7 lb

## Hatch Cover

Loading: 20 psi over assumed area of 5.25 ft sq

$$w = \frac{20 \times 144}{1} = 2880 \ lb/ft \qquad L = 5' \ assumed$$

Note: Assume fixed end beam (bolted connection)

Cover Stringers:

$$M = \frac{wL^2}{12} = \frac{2880 \times (5.00)^2}{12} = 6000 \ ft\text{-}lb$$

Shear:

$$V = \frac{wL}{2} = \frac{2880 \times 5.00}{2} = 7200 \ lb, \ b = 12"$$

$$h = \frac{3V}{2bH} = \frac{3 \times 7200}{2 \times 12 \times 640} = 1.41"$$

Bending:

$$h^2 = \frac{6M}{bf} = \frac{6 \times 12 \times 6000}{12 \times 6000} = 6.00 \text{ in.}^2$$

$$h = 2.45"$$

$$\text{Beam Width} = \frac{V}{c_{\perp} \times b} = \frac{7200}{1100 \times 12} = 0.545" \text{ minimum}$$

Use 8" x 3" x 5' - 4-1/4" = 11 fbm and 26 lb          8 required

## Beam and Siding:

Beam is supported by sill for hatch cover. Beam is 3" wide and 5'-0" long, h = 4", siding between beams would be 4" x 3" x 5'-4-1/4".

Beam each = 4 fbm and 12 lb
Siding each = 4 fbm and 12 lb

## Sills:

Total load on hatch cover is 20 x 144 x (5.27 x 5.35) = 77,000 lb

$$\text{Sill load} = \frac{77,000}{2} = 38,500 \text{ lb}$$

$$\text{Sill size} = \frac{38,500}{2000} = 19.25 \text{ ft}^2 \text{ when soil bearing} = 2000 \text{ psf}$$

Obviously, this would be an immense sill. Using the bearing strength of medium clay (8000 lb) would result in a more reasonable sill.

$$\text{Area} = \frac{39,650}{8000} = 4.96 \text{ ft}^2$$

Considering the time element (6 sec. total positive phase) of the 20-psi blast wave from a 20-MT weapon and the slow settlement of sills, it seems reasonable to use a medium bearing strength (Reference 1, App. B).

A 12" wide timber sill area = $\frac{11.5}{12}$ x 5.50 = 5.04 ft$^2$

Deadman:

For restraining hatch cover against uplift caused by negative pressure.

Maximum negative pressure = 0.13 x 20 (Reference 11, App. B)

Maximum negative pressure = 2.6 psi

Total uplift = $P_{neg.}$ x A = 2.6 x 5.5 x 5.5 x 144 = 11,300 lb

Deadmen placed below footings at 30-degree angle from the vertical. Deadmen are 5 ft vertically below footing. Deadman is attached to footing by tie rod.

Length of tie rod = $\dfrac{5}{\cos 30^{\circ}}$ = $\dfrac{5}{0.866}$ = 5.78'

Tensile load on tie rod = $\dfrac{11,300}{2 \cos 30^{\circ}}$ = $\dfrac{5650}{0.866}$ = 6520 lb

Assume soil resistance against liftup of deadman = 3000 psf

Area of deadman = $\dfrac{6520}{3000}$ = 2.17 ft$^2$ required

Length of deadman = 5.5'          Width = $\dfrac{2.17}{5.5}$ = 0.40' min.

Use width of 6" nominal, 5-5/8" actual

Tie Rod:

R = P = 3260        $f_s$ = 50,000 psi (steel)

$A_s$ = area of steel = $\dfrac{P}{f_s}$ = $\dfrac{3260}{50,000}$ = 0.0652 in.$^2$

Radius of rod = $\dfrac{\sqrt{A_s}}{\pi}$ = $\dfrac{\sqrt{0.0652}}{3.1416}$ = $\sqrt{0.0208}$ = 0.1444

Use 5/16" round rods, $A_s$ = 0.805 in.$^2$

Connectors:

$c_{\perp}$ = 1100, area of washer = $\dfrac{P}{c_{\perp}}$ = $\dfrac{3260}{1100}$ = 2.97 in.$^2$

Add area of hole in washer, $A_w = 2.97 + \dfrac{\pi(3)^2}{(16)^2} = 3.08$

Radius $= \dfrac{\sqrt{3.08}}{\pi} = \sqrt{0.98} = 0.99"$      use 2" dia. washer

Exhibit 2

METAL STRUCTURES

PIPE ARCH SHELTER CALCULATIONS

Problem

What gage no. is necessary on a 10' 8" span 6' 11" rise Pipe Arch
Sectional Plate Structure under certain given conditions.

Given:

    1.    Five-foot earth cover.

    2.    Design for total static loading of 7 psi (includes dead
        load from earth cover).

    3.    Use safety factor of "4".

210
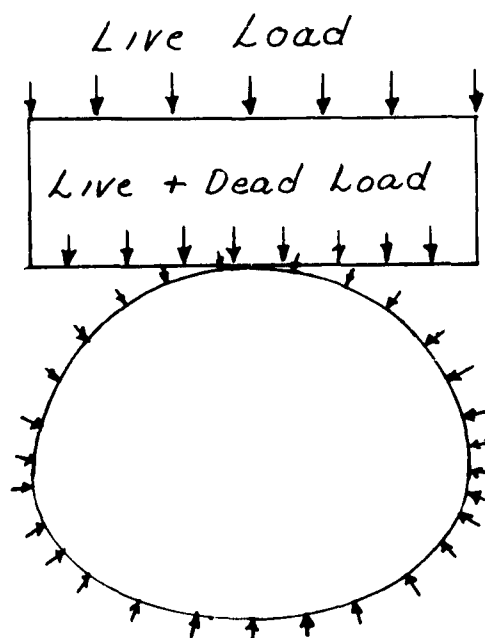
Solution:

1.  Seam Strength:

$$C = \frac{PS}{2}$$

Where:  C = Compressive Strength
        P = Live plus dead load
        S = Span

$$C = (7)\frac{(128)}{2} = 448 \text{ lb/in., or}$$

C = 5376 lb/foot.

Considering a safety factor of 4:

C = 4 x 5376 = 21,504 lb/ft

From Reference 5, App. C, pp. 50, Fig. 41, 14 gage Sectional Plate with 3/4-in.-diam. high-tensile bolts spaced 4 in. apart on centers at all seams, is required.

2.  Pipe Arch Strength:  Determine pipe arch cover equivalent to 7 psi (1008 psf) total load:

$$\text{Vol.} = (\frac{128}{12})(1)(H_e) = \frac{32}{3} H_e$$

$$\text{Load Area} = (\frac{128}{12})(1) = \frac{32}{3} \text{ ft}^2;$$

$$\text{at } 1008 \text{ lb/ft}^2, (\frac{32}{3})(1008) \text{ lb}$$

$$\text{If ave. soil} = 100 \text{ lb/ft}^3, \text{ Vol.} = (\frac{32}{3})(\frac{1008}{100}) \text{ ft}3$$

$$\therefore \quad H_e = (\frac{32}{3})(\frac{1008}{100})(\frac{3}{32}) = \underline{10.08 \text{ ft}}$$

Assume $H_e$ = 10 ft

From Reference 5, App. C, pp. 115, Table 9-12 at 10 ft, the gage required is No. 10. This table includes a safety factor of 4. Therefore, calculation No. 2 rules and the pipe arch shall be No. 10 gage Sectional Plate, 10' 8" span,

6' 11" rise, with 3/4-in. high-tensile bolts spaced every 4 ft at all seams.

3. Weight and Handling Ability:

Weight in pounds per lineal foot of structure is: 249 pounds. It is made up of two 9 Pi, one 15 Pi, three 18 Pi, and one 21 Pi sections per ring. Sections are standard and 6- and 8-ft lengths. Therefore, weight per section is:

|  | 9 Pi | 15 Pi | 18 Pi | 21 Pi |
|---|---|---|---|---|
| 6 ft | 124.56 lb | 207.6 lb | 249.12 lb | 290.64 lb |
| 8 ft | 164.08 lb | 276.8 lb | 332.16 lb | 387.52 lb |

The larger above sections require special handling equipment such as an "A" frame.

In large quantities, it is feasible to manufacture the Sectional Plate in shorter lengths of say 2 and 4 ft. The weights per plate in these lengths would be:

|  | 9 Pi | 15 Pi | 18 Pi | 21 Pi |
|---|---|---|---|---|
| 2 ft | 41.52 lb | 69.2 lb | 83.04 lb | 96.88 lb |
| 4 ft | 82.04 lb | 138.4 lb | 166.08 lb | 193.76 lb |

In these lengths the sections could be handled without the use of an "A" frame and are, therefore, recommended.

4. Maximum Load Capabilities:

Safety factor of 4.

4 x 7 = 28 psi total load. 5 ft of cover is equivalent to:

$$P_5 = \frac{[(128)]\,[(5)(12)]\,[100]}{[(128)]\,[(144)(12)]} = \underline{3.47\ psi}$$

Therefore, the shelter could withstand an additional (28 - 3.47) = 24.53 psi above an earth loading of 5 ft.

## ENTRANCEWAY UNDERPASS CALCULATIONS

I. Rough Sketch:



5'9" Span   6'6" Rise
Underpass type

3' dia. Cor. pipe

Hatch cover
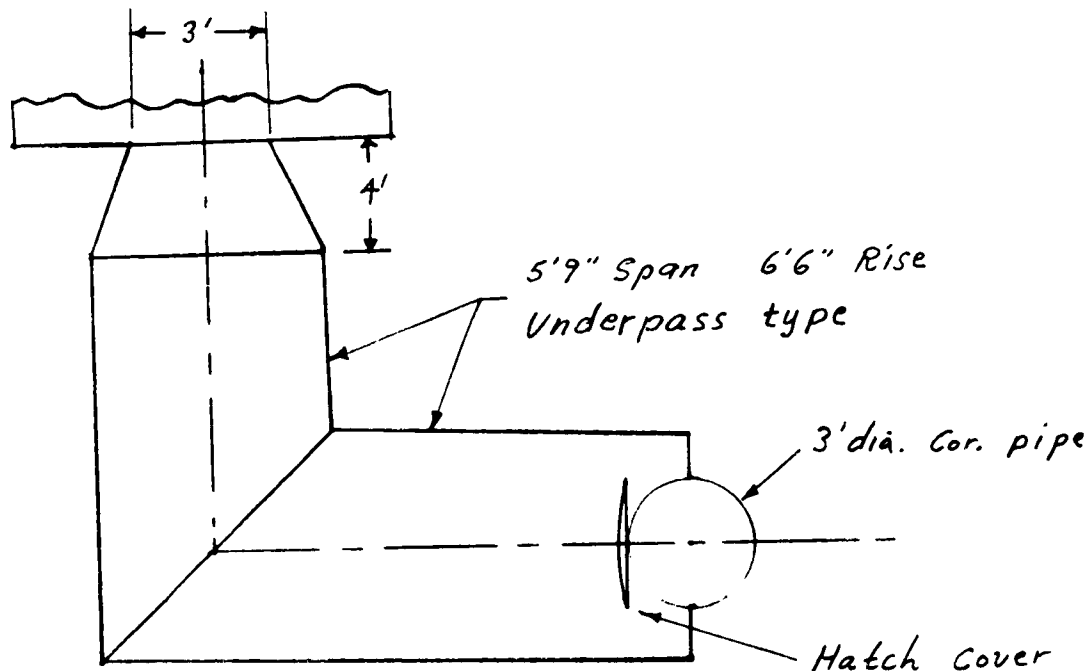
II. Determine gage sizes for underpass:

A. Seam Failure:



$w = 100$ lbs/ft
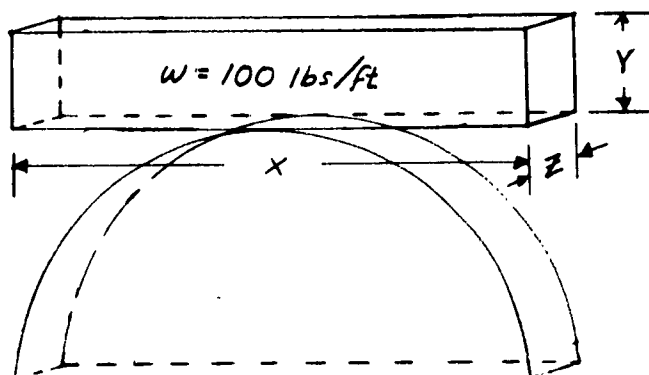
$C = P \dfrac{S}{2}$

Consider $S = 6'$, then $C = 3P$

$P = $ psi loading $= 7$ psi

$C = 3(7)(12) = 252$ lb/in.

$C = 3024$ lb/ft

From pp 50, Reference 5, App. C, proper gage is 12 ga. or even lighter using 3/4-in.-dia. high-tensile steel bolts spaced 3 in. apart and double rows in the longitudinal seams.

B.  Failure of Sectional Plate:

Rise = 6' 6", Span = 5' 9"

Consider as a 6' diam. pipe for design purposes.

(1) Equivalent Earth Cover to a 7-psi static loading:
    See Fig. 3, Appendix C.

Consider 100 lb/ft$^3$ density soil.  Y = ?

Vol. = XYZ;  load = F = pA = pXZ;

$$Vol. = \frac{load}{soil\ density} = \frac{pXZ}{100}$$

$$XYZ = \frac{pXZ}{100} \qquad where\ p = lb/ft^2$$

$$Y = \frac{p}{100} = \frac{7(144)}{100} = \underline{10.08\ ft}$$

<u>Use 10 ft.</u>

The only load vs. gage tables available for commercially available sectional plate are those for H-20 loadings plus specific soil coverings.  There are none for soil covering alone.  With an H-20 live load and 10 ft of cover, 12 ga. sectional plate is necessary for the size used.  This would then include a "built in" safety factor due to the unused H-20 live loading capability.  Also, the tables used were constructed using a safety factor of 4.  Therefore, full shelter capability would be:

4x7 = 28 psi plus H-20 live load capability

C.  Actual Selection:  Considering "A" and "B" the actual selection is for a 6' 6" rise, 5' 9" span, 12 ga. underpass-type structure.

III. Vertical Hatchway Pipe Gage Determinations:

Consider (1) 36" diam. pipe.
         (2) 5 psi design stress

A. Seam Failure:

$p = 5\ psi$
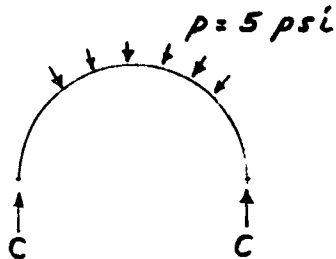
$$C = P(\tfrac{S}{2}); \quad \text{where } S = \text{diam.}$$
$$P = \text{pressure}$$

$$C = (5)(144)(\tfrac{3}{2}) = 5(216)$$

$$C = \underline{1080\ \text{lb/ft}}$$

From p. 50, Fig. 41 of Reference 5, App. C, interpolating, 16 ga. would be sufficient.

B. Collapsing Strength:

(1) Equivalent feet of cover to a 5-psi static load:

Considering 100 lb/ft$^3$ density soil.

$$L = \frac{P(144)}{100}$$

$$L = \frac{5(144)}{100} = \underline{7.2\ \text{ft.}}$$

From mfg's tables, a 12-ga. pipe with 7 ft of cover plus an H-20 live load is sufficient. The tables were constructed with a safety factor of 4. Therefore, the ultimate pipe capability would be:

4x5 = 20 psi plus the unused H-20 loading capability.

Therefore, the selection is for 36-in.-diam. corrugated pipe of 12-gage steel.

Underpass Weights, Lengths, and Widths

1. Entranceway weight per foot = 184 lb

2. Sect. Plate Widths:

a. Two corner - 9 Pi = 2.355 ft = 28.3 in. for a total of
2 x 2.355 = 4.710 ft.

b. Two side - 15 Pi = 47.15 in. = 3.93 ft for a total of
2 x 3.93 = 7.86 ft.

c. One top - 21 Pi = 66 in. = 5 ft 6 in.

d. One bottom - 9 Pi = 28.3 in. = 2.355 ft

3. Sect. Plate weights; 12 ga. Sect. Plate:

| Width | Length | | | |
|---|---|---|---|---|
| | 2 ft | 4 ft | 6 ft | 8 ft |
| 9 Pi | 42.46 lb | 84.92 lb | 127.38 lb | 169.84 lb |
| 15 Pi | 70.76 lb | 141.52 lb | 212.28 lb | 323.04 lb |
| 21 Pi | 99.08 lb | 198.16 lb | 297.24 lb | 396.32 lb |

The 2-ft and 4-ft length combination can be handled without an "A" frame or other equipment and is, therefore, recommended.

## HATCHWAY COVER BRACING DESIGN

Given:  $p = 28$ psig

Determine $w_1$, $w_2$, $w_3$:

$$F_T = A_T p = (A_1 + A_2 + A_3)p = F_1 + F_2 + F_3$$

$$F_T = (28)(\pi)(24)^2 = 33,057.92 \text{ lb}$$

$$F_1 = F_2; \quad \frac{F_1}{F_3} = \frac{A_1}{A_3} \quad \text{or} \quad F_1 = F_3 \left(\frac{A_1}{A_3}\right)$$

$$A_1 = A_2 = 4 \times 48 = \underline{192 \text{ in.}^2}$$

$$A_3 = \pi(r_o^2 - r_1^2) = \pi[(24)^2 - (18)^2] = \pi(252)$$

$$A_3 = \underline{791.28 \text{ in.}^2}$$

$$\therefore A_T = \underline{1175.28 \text{ in.}^2}$$



Circular Area = $A_3$
48" O.D.
36" I.D.

$$F_3 + \left(\frac{192}{791.28}\right) F_3 (2) = 33,057.92$$

$$F_3 + .486 \, F_3 = 33,057.92$$

$$\therefore F_3 = \frac{33,057.92}{1.486} = \underline{22,250 \text{ lb}}$$

$$\therefore F_1 = F_2 = (.243)(22,250) = \underline{5,410 \text{ lb}}$$

$$w_1 = w_2 = \frac{5410}{.5} = \underline{10,820 \text{ lb/ft}}$$

$$w_3 = \frac{22,250}{4} = \underline{5,562.5 \text{ lb/ft}}$$

Load, Shear, and Moment Diagrams

48"

$W = 10,820\ lbs./Ft.$

6"

6"

$W = 43,280\ lbs/Ft.$        $W = 43,280\ lbs./Ft.$

$V_{max} = 16,230\ lbs$

6"    18"

8344

Shear

$M_{max} = (1/2)(1/2)\ 16,230$
$+ (1/2)(3/2)\ 16,230$
$M_{max} = 16,230\ ft.\text{-}lbs.$

$M = 16,230\ ft.\text{-}lbs.$

Moment
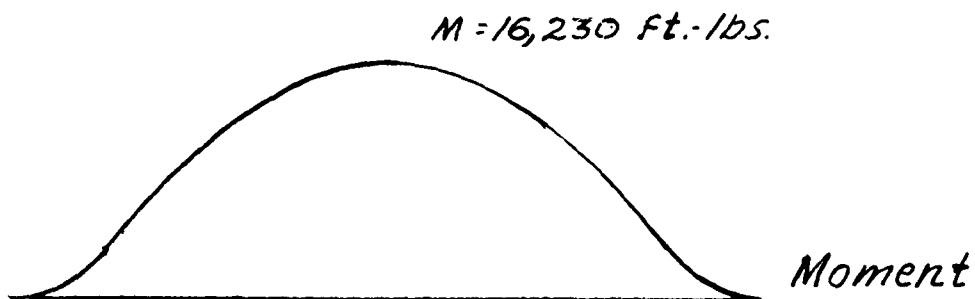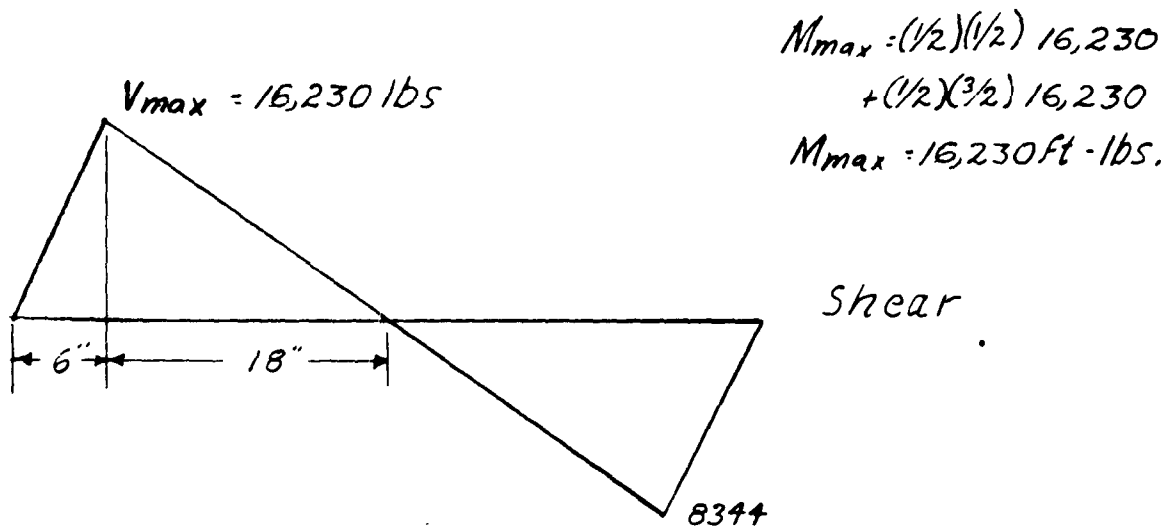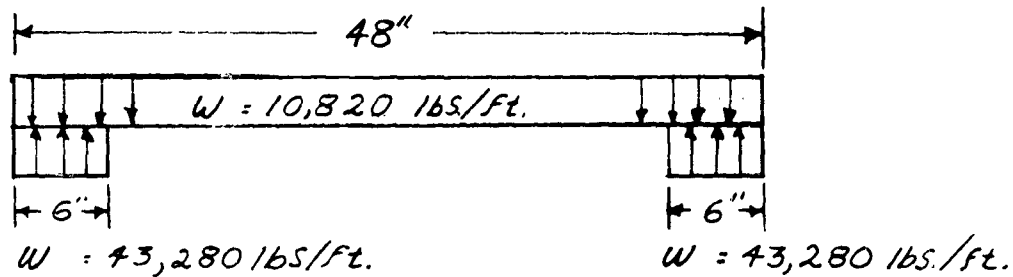
$$S_{B_{max.}} = \frac{M_{max.}}{Sect.\ Mod.} = \frac{M_m}{Z}$$

Consider $S_u$ for welded structural alum. joint = 34,000 psi in tension.

$$Z = \frac{M_m}{S_{B_{max.}}} = \frac{(8344)(12)}{34,000}$$

$$Z = \underline{0.295\ in.}^4 \qquad \text{minimum required.}$$

$$S_{S_{max.}} = \frac{3}{2}\ \frac{V}{A_{S_{min.}}} \qquad\qquad A_{S_{min.}} = \frac{3}{2}\ \frac{V}{S_{S_{max.}}}$$

$$A_S = \left(\frac{3}{2}\right)\ \frac{(8344)}{16,000}$$

Consider $S_u$ for welded structural alum. joint = 16,000 psi in shear.

$$A_S = \underline{.782\ in.}^2 \qquad \text{minimum required.}$$

Possible Choices:   Reference 7, App. C.

  (1)  4 H 4.85,      $Z = 5.36$ in.$^4$ (smallest avail.);  $A_S = 4.0$ in.$^2$

  (2)  6 x 4 WF 4.28, $Z = 7.25$ in.$^4$ (smallest avail.);  $A_S = 3.54$ in.$^2$

*(3)  5 [ 2.38,      $Z = 3.00$ in.$^4$;  $A_S = 1.97$ in.$^2$

  (4)  4 I 2.72,      $Z = 3.03$ in.$^4$;  $A_S = 2.25$ in.$^2$

  (5)  5   4.84,      $Z = 2.90$ in.$^4$;  $A_S = 4.00$ in.$^2$

  (6)  4 T 5.56,      $Z = 3.14$ in.$^4$;  $A_S = 4.60$ in.$^2$

  * Final choice, as most economical

Note:  This design is such that cover rests on earth, not corrugated material. This is so as not to stress the corrugated hatchway in its weakest direction, longitudinally.

Cover Weight:

$$\frac{Wt\ of\ Al.}{Wt\ of\ Steel} = \frac{.1}{.281} = \frac{1}{2.81}$$

Wt of steel (Reference 5) for Sect. Plate 12 ga.

$$lb/ft^2 = \frac{278(12)}{21(\pi)(8)} = 6.31\ lb/ft^2$$

218

A1. $wt = \dfrac{6.31}{2.81} = 2.25 \ \text{lb/ft}^2$

$Area = 4\pi$

$Wt = 4\pi(2.25) = 9\pi = 28.27 \ \text{lb} = \underline{29 \ \text{lb}}$

## BULKHEAD DESIGN FOR PIPE ARCH BOMB SHELTER

The bulkhead is designed for a total horizontal loading of 20 psi or 2880 psf, and has a span of 10' 8" and a rise of 6' 11". It is assumed that the bulkhead will take negligible vertical loading. The material used is 2014-T6 aluminum which has the following properties:

$f_u$ (ultimate strength)          70,000 psi

$f_y$ (yield strength at .2% set)     60,000 psi

t (shear strength)             42,000 psi

The members of the bulkhead are allowed to deform plastically under the dynamic loading. The ductility ratio ($\beta$) used is 5.5, which should allow the structure to be subjected more than once to the design loading. The dynamic load factor (b) is a function of the ductility ratio and is approximated as follows:

$$\frac{2 - b}{2b - 2} = \beta - 1$$

It is assumed that the loading is instantaneous and constant to the point of maximum deflection.

When $\beta = 5.5$, $b = 1.1$ the equivalent static load is computed as follows:

$$p_s = b p_d$$

$p_s$ = Equivalent static load

b  = Dynamic load factor

$p_d$ = Dynamic load

When a beam deforms plastically in bending

$$M_s = F \frac{f_{dy} I}{c}$$

$M_s$ = static moment

F  = Shape factor of beam

$f_{dy}$ = Dynamic yield strength

$$f_{dy} = (f_u + f_y)/2 = 65,000 \text{ psi} \qquad 64,000 \text{ used in design}$$

$$F \sim \frac{c \int_0^c y dA}{\int_0^c y^2 dA} \quad \text{for wide flange and standard beams and channels.}$$

$$Ms = M_d b \qquad M_d = \text{Moment computed using } P_d$$

It then follows that:

$$Md = \frac{F}{b} \frac{f_{dy} I}{c}$$

For wide flange and standard beams, F may be taken as 1.1 and since b = 1.1

$$M_d = \frac{f_{dy} I}{c}$$

The required section modulus (S) is then

$$S = \frac{I}{C} \geq \frac{M_d}{f_{dy}}$$

For axially loaded members in tension, the required area (A) is

$$A = \frac{b P_d}{f_{dy}}$$

Since these members are loaded by reactions and not by dynamic loads directly, the dynamic load factor may be reduced. In this case, the value of b approaches 1.0. Using this value, the required area is

$$A = \frac{P_d}{f_{dy}}$$

The bulkhead is designed so that failure will probably first occur in the corrugated aluminum sheeting.

Consider 2014-T6 aluminum corrugated sheeting   $f_u = 64,000$ psi

pitch = 6"      depth = 2"      use 5 gauge      $S = .1147$ in.$^3$/in.

$w = 2880$ lb/ft$^2$      $M_{allow} = .1147 \times 64,000 = 7350$ ft lb/ft

$$\frac{wL^2}{8} = 7350 = \frac{2800\ L^2}{8} \qquad L^2 = 20.4 \qquad L = 4.51' \quad \text{simply supported}$$

$$\frac{wL^2}{2} = 7350 = \frac{2880\ L^2}{2} \qquad L^2 = 5.1 \qquad L = 2.25 \quad \text{cantilever}$$
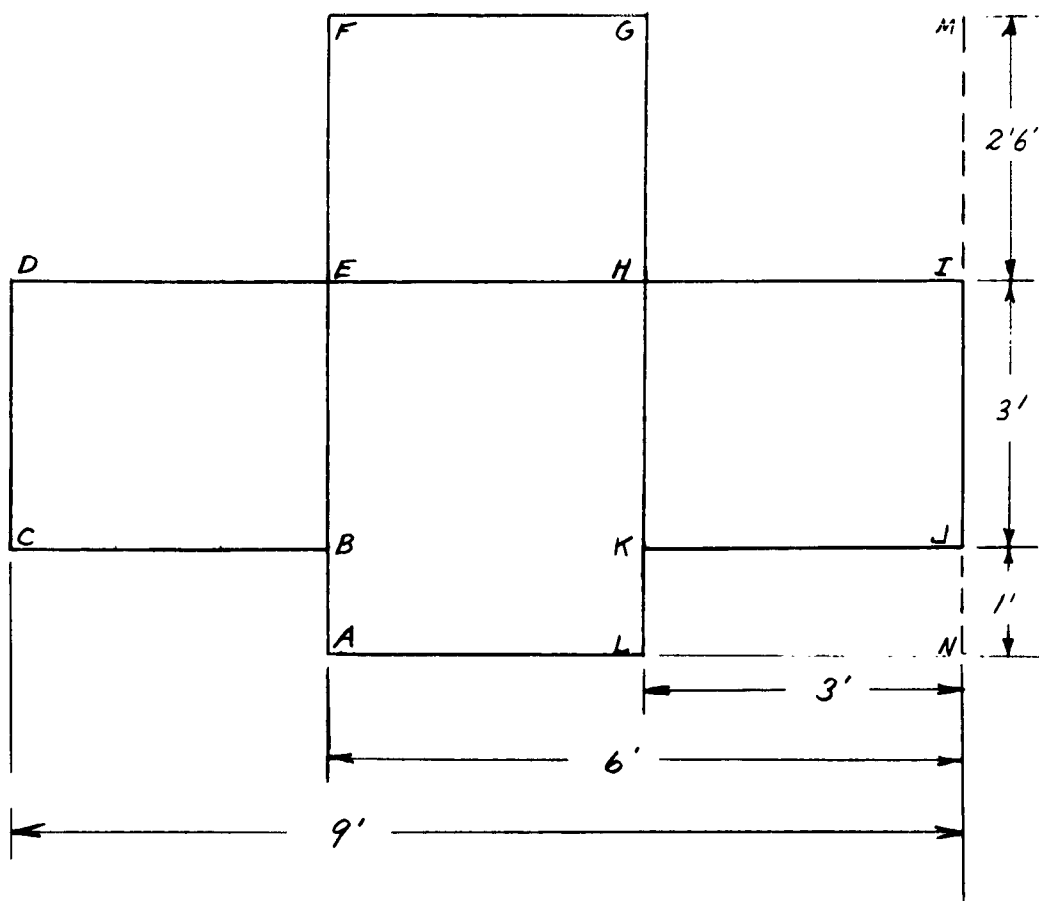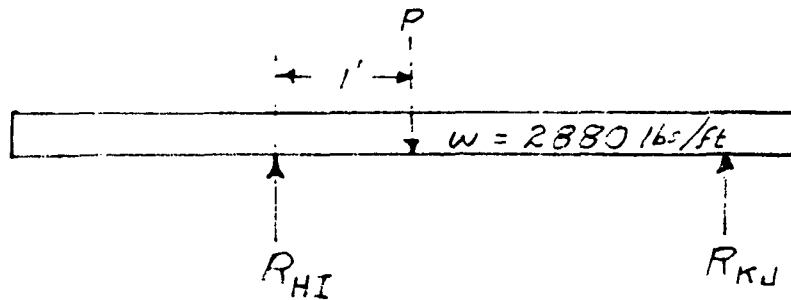
$$w = 2880 \times 5 = 14{,}400 \text{ lb}$$

$$3R_K = 14{,}400$$

$$R_K = 4{,}800 \text{ lb/ft}$$

$$R_h = 9{,}600 \text{ lb/ft}$$
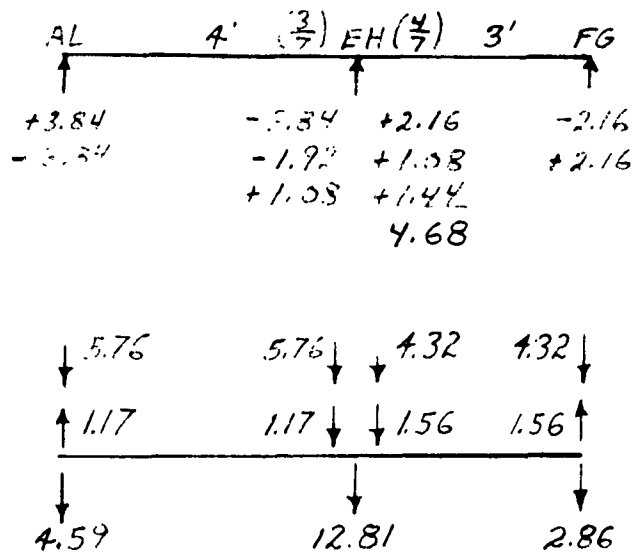




Member FG    $M = 2.86 \times \frac{9}{8} = 3.22$ k'    S.M. $= 3.22 \times 12/64 = 0.604$,
$W = 8.57$

Member EH    $M = 12.81 \times 1.125 = 14.5$ k'    S.M. $= 14.5 \times 12/64 = 2.72$,
$W = 38.6$

Member AL    $M = 4.59 \times 1.125 = 5.17$ k'    S.M. $= 5.17 \times 12/64 = 0.97$,
$W = 13.75$

Member HI    M = 9.6 x 1.125 = 10.8 k'   S.M. = 10.8 x 12/64 = 2.03,
           W = 32.4

Member KJ    M = 4.8 x 1.125 = 5.4 k'   S.M. = 5.4 x 12/64 = 1.02,
           W = 14.4

Members LG and AF;

     Load at joint H = 1/2(32,400 + 38,600) = 35,500 lb
     Load at joint K = 1/2(14,400) = 7,200 lb
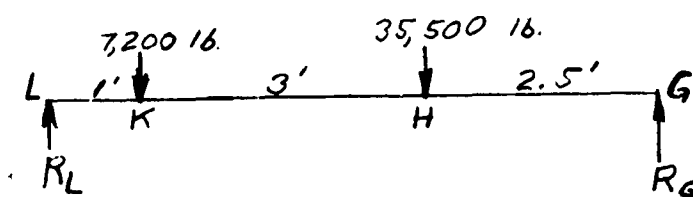


$\Sigma M_L = 0;$

6.5 $R_G$ = (1)(7,200) + 4(35,500) = 149,200 ft-lb

        $\underline{R_G = 23,000 \text{ lb}}$

$\Sigma F_Y = 0;$   7,200 + 35,500 - 23,000 = $R_L$ = 19,700 lb.

Max. Moment = $M_{Max.}$ = 2.5 (23,000) = 57,500 ft-lb

Sect. Mod. = S = $\dfrac{\text{Max. Mom. x 12}}{\text{Max. Tensile Stress}}$

     S = $\dfrac{57,500}{64,000}$ x 12 = 10.8 in.$^3$, minimum required sect. modulus.

Possible beam selections are:

     7 I 6.23       S = 11.26      t = 345
     8 ⸦ 6.67       S = 10.99      t = .190
     8 WF 5.90     S = 14.18      t = .230

Use:   8 WF 5.90 for beams LG and AF as both are similarly loaded.

Member EH;  (S required = 2.72)

    Possible choices:

    4 I 2.64      S = 3.03      t = .190
    5 [ 2.32      S = 3.00      t = .190
    5 [ 2.50      S = 3.14      t = .225

    Use 4 I 2.64 for EH

Members HI and DE;  (S required = 2.03)

    Possible choices:

    4 I 2.64      S = 3.03      t = .190
    4 [ 2.16      S = 2.10      t = .247
    4 [ 2.50      S = 2.29      t = .320

    Use:  4 I 2.64 for beams HI and DE

Member AL;  (S required = 0.97)

    Possible choices:

    3 [ 1.42      S = 1.10      t = .170
    3 [ 1.48      S = 1.13      t = .187
    3 [ 1.73      S = 1.24      t = .258

    Use:  4 [ 2.16, although lighter can be used, in order to
          minimize number of beam sizes required.

Member FG;  (S required = .604)

    Same as beam AL (4 [ 2.16)

Members KJ and CB;  (S required = 1.02)

    Possible choices:

    3 [ 2.02      S = 1.68      t = .170
    3 [ 1.42      S = 1.10      t = .170
    3 [ 1.48      S = 1.13      t = .187
    3 [ 1.73      S = 1.24      t = .258

    Use:  4 I 2.64 for beams KJ and CB, although lighter can be used,
          in order to minimize number of beam sizes required.

Members CD and IJ:  (S required = O)

These members were considered as not carrying any load.  There-fore, any choice can be used.  Use 4 [ 2.16 for beams CD and IJ at rear of shelter, and at front except use 8 WF 5.90 for beam IJ (extended) at front in order to allow clearance for deadman tie-in over entrance way.

Bearing check on heaviest loaded beams:

Member EH:

$$R_E = 19,300 \text{ lb, } A = \frac{F}{S} = \frac{\text{Force}}{\text{Tensile limit}}$$

$$A = \frac{19,300}{64,000} = .302 \text{ in.}^2$$

$t_1 = .190, L_1 = 1.59; \quad t_2 = .225, L_2 = 1.34$

Use two 7/8-in. diam bolts, and 1/4-in. thick connecting angles.

Members AL and FG:

$$R_A = 6,870, A = \frac{\text{Force}}{\text{Tensile Limit}} = \frac{F}{S}$$

$$A = \frac{6,870}{64,000} = .107 \text{ in.}^2$$

$$L = \frac{A}{t} = \frac{.107}{.247} = \text{approx. } 1/2\text{-in.}$$

Use two 5/8-in. diam. bolts.

Determination of deadman tie rod diameter:

Load at D, I;

$$1/2 (32,400) = 16,200 \text{ lb}$$

$$\text{Area} = \frac{\text{Force}}{\text{Stress Limit}} = \frac{16,200}{64,000} \text{ and approx. } 1/4 \text{ in.}^2$$

$$.7845d^2 = .25$$
$$d = .562$$

Use 5/8-in. diam. Al rod at points D and I

Load at C, J;

$$1/2 \ (14,400) = 7,200 \ \text{lb}$$

This is less than at joints "D" and "I", therefore 5/8-in. diam. Al rod is satisfactory for joints "C" and "J".

Load at F, G;

$$R_G = R_{GL} + R_{FG}$$

$$R_G = 23,000 + 4,300 = 27,300 \ \text{lb}$$

$$A = \frac{F}{S} = \frac{23,000}{64,000} = .427 \ \text{in.}^2$$

$$d^2 = .544, \ d = .738$$

Use 7/8-in. diam. Al rod.

Load at A, L;

$$R_A = R_L = R_{LG} + R_{LA}$$

$$R_L = 19,700 + 6,900$$

$$R_L = 26,600 \ \text{lb.}$$

This is less than load on joints "G" and "F", therefore 7/8-in. diam. Al rod is satisfactory for joints "A" and "L".

Check on joint loads:

Total Load = Total area x psf

$$L = (58)(20)(144) = 167,000 \ \text{lb}$$

$$16,200 + 7,200 + 27,300 + 26,600 = 77,300 \ \text{lb}$$

$$77,300 \ \text{x} \ 2 = 154,600 \ \text{lb}$$

$$\underline{154,600} \ \sim \ \underline{167,000}$$

This approximation appears satisfactory in consideration of rod diam. used in comparison to diam. required for tie rods.

Weight of bulkhead:

Beam Weight:

| | | |
|---|---|---|
| AF and GL: 2(6.5)(5.90) | | = 76.6 |
| DE, EH, HI, CB, and KJ: 5(3)(2.64) | | = 39.6 |
| DC and IJ: 2(3)(2.16) | | = 12.96 |
| Total Beam Weight | | = 129.16 lb |

Corrugated sheet weight:

$$Wt = (4.8 \text{ lb/ft}^2)(58 \text{ ft}^2)$$

$$Wt = 280 \text{ lb}$$

Approximate connection weight is 40 lb. Therefore, total bulkhead weight is estimated at 450 lb.

Beam Plan
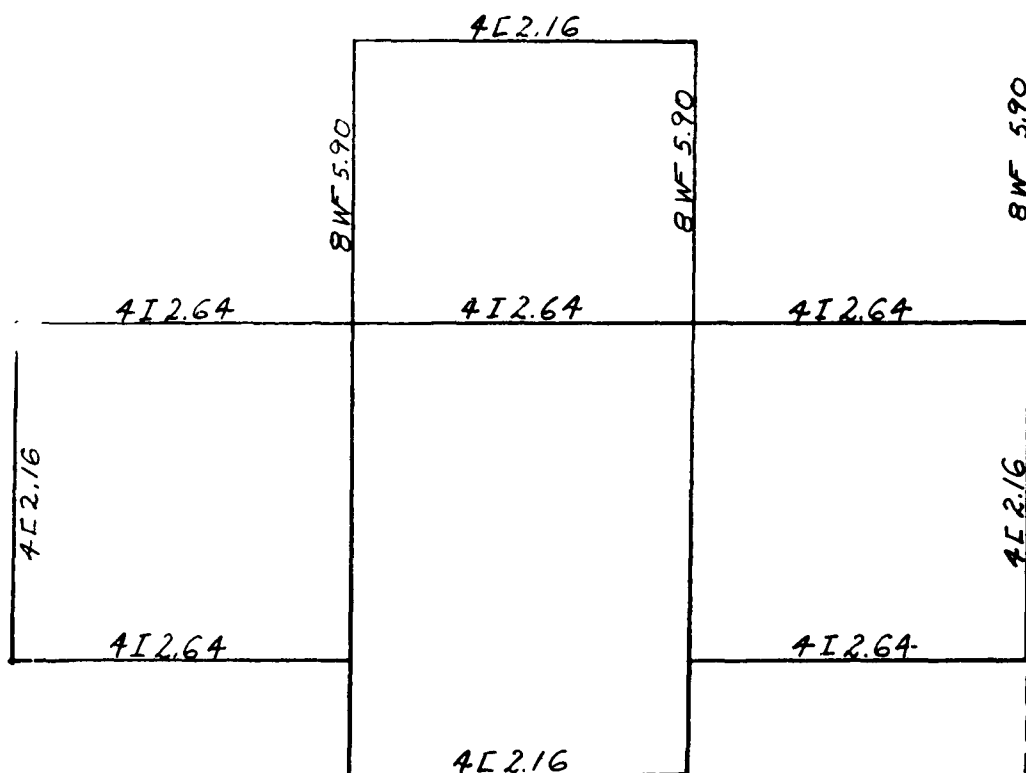
Determining dynamic load factor for a beam.

Assume:

1. The dynamic load is constant.

2. The mass of the beam is concentrated at the center of the beam.

3. That the beam deflects elastically to the point where the maximum fiber stress equals the dynamic yield stress. Further deflection is assumed to be totally in the plastic region.

b = dynamic load factor = $P_s/P_d$      $\beta$ = ductility ratio

$P_d$ = dynamic load      $\delta_2$ = maximum deflection

$P_s$ = equivalent static load      $\beta$ = $\delta_2/b\delta_1$

K = spring constant of beam

$\delta_1$ = $P_d/K$

W = weight of beam

m = mass of beam

g = 32.2

x = deflection

$\omega^2$ = $K_g/W$

For elastic region;

$$m\ddot{x} + Kx = P_d$$

Solution of differential equation yields:

$$x_1 = \delta_1 - \delta_1 \cos \omega t \quad \text{for} \quad 0 < x < b\delta_1$$

at $x_1 = b\delta_1$,

$$b\delta_1 = \delta_1 - \delta_1 \cos \omega t; \quad \cos \omega t = 1-b, \quad \text{Sin } \omega t = (2b - b^2)^{1/2}$$

$$\dot{x}_1 = \delta_1 \omega \sin \omega t = \delta_1 \omega (2b - b^2)^{1/2} \quad \text{at} \quad x = b\delta_1$$
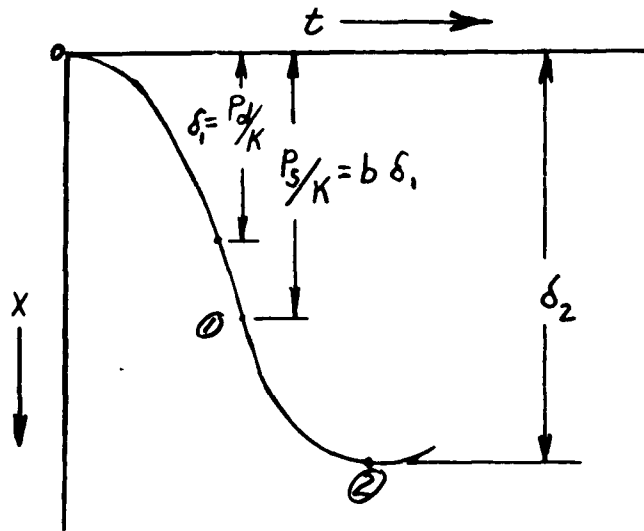
going into plastic region,

$$m\ddot{x} + Kb\delta_1 = P_d \qquad\qquad \text{Since } \dot{x} \text{ is constant,}$$

$$m\ddot{x} = P_d = Kb\delta_1 \qquad\qquad \dot{x}_2^2 - \dot{x}_1^2 = 2\ddot{x}\,(x_2 - x_1)$$

$$\ddot{x} = \frac{P_d - Kb\delta_1}{m} \qquad\qquad \dot{x}_2 = 0$$

$$-\delta_1^2\,\omega^2\,(2b - b^2) = 2\,\frac{g}{W}\,K\delta_1\,(1 - b)\,(\beta - 1)\,b\delta_1$$

$$-(2b - b^2) = 2(1 - b)\,b\,(\beta - 1)$$

$$\frac{2 - b}{2b - 2} = \beta - 1$$

Effect of changing b on β

| b | B | b | β |
|------|------|-----|------|
| 1.00 | ∞ | 1.1 | 5.5 |
| 1.01 | 45.5 | 1.2 | 3 |
| 1.02 | 25.5 | 1.3 | 2.12 |
| 1.03 | 17.1 | 1.4 | 1.75 |
| 1.04 | 13.0 | 1.5 | 1.5 |
| 1.05 | 10.5 | 1.6 | 1.33 |
| 1.06 | 8.8 | 1.7 | 1.21 |
| 1.07 | 7.6 | 1.8 | 1.12 |
| 1.08 | 6.8 | 1.9 | 1.06 |
| 1.09 | 6.0 | 2.0 | 1.0 |

Determination of shape factor, $F = F = S_p/S_e$
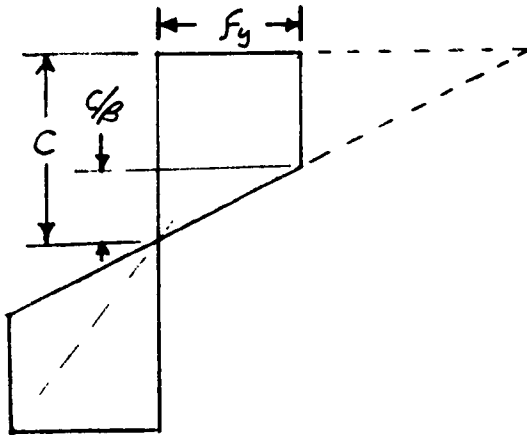
Where $S_p$ = Effective section modulus in plastic range

$S_e$ = Section modulus in elastic range

$$1/2\ S_p = \beta/C \int_0^{C/\beta} y^2\ dA + \int_{C/\beta}^0 y\,dA$$

$$1/2\ S_e = 1/c \int_0^{C/\beta} y^2\ dA$$

$$F = \frac{\beta \int_0^{C/\beta} y^2\ dA + C \int_{C/\beta}^C y\,dA}{\int_0^C y^2\ dA}$$

When $\beta = 1$, $F = 1$

When $\beta = \infty$, $F = \dfrac{c \int_0^C y\,dA}{\int y^2\ dA}$

## METAL-TIMBER STRUCTURES

The arch-shaped shelter is designed for a load of 23.5 psi. A design stress for steel in bending of 50,000 psi is used. The following allowable stresses are used for wood:

| | |
|---|---|
| bending | 6000 psi |
| horizontal shear | 640 psi |
| compression parallel to grain | 4800 psi |
| compression perpendicular to grain | 1100 psi |

Wood members in bending, loaded directly by the dynamic load, are designed using a dynamic load factor of 1.5.

The shelter is 16' wide and 40 ft long.

Consider arches at 4' center to center.



$P = 23.5 \times 144 = 3380 \text{ lb/ft}^2$
$W = 3380 \times 4 = 13,530 \text{ lb/ft}$

$A_v = 8 \times 13.53 = 108.2 \text{ kips}$     Area $= 108.2/50 = 2.16 \text{ in.}^2$

Use ST 4 WF 12     Area $= 3.53 \text{ in.}^2$

Check bending in flange

$t_f = .398 \text{ in.}$     $S = \frac{12}{6} \times .16 = .32$

$M = 1.5 \times 6.76 = 10.2 \text{ in.-kips}$     $S_{req.} = \frac{10.12}{50} \sim .2$

on ends of shelter use a 4" x 3" x 1/2" angle

## Timber Beams:

$M = \frac{WL}{8} = 13.53 \times \frac{4}{8} = 6.76 \text{ kip-ft}$     $S/ft = 6.76 \times \frac{12}{6} = 13.52 \text{ in.}^3$

$$\frac{d^2}{6} = \frac{13.52}{12} \qquad d^2 = 6.76$$

$$d^2 \times D.L.F. = 6.76 \times 1.5 = 10.12 \qquad d = 3.2''$$

Use 4" x 8" x 3' 11" timber

Check for shear

$$A = 3.625 \times 12 = 43.5 \qquad V/A = 6.76/43.5 = 155 \text{ psi}$$

required bearing length

$$L = \frac{6760}{1100 \times 12} \sim 1/2 \text{ inch}$$

Bearing Plates for Arches:

$$\text{required area} = \frac{108.2}{1.1} = 98.3 \text{ in.}^2$$

Use 6" x 8" x 1" plate

$$\sigma = \frac{108.2}{48} = 2250 \text{ psi}$$

This stress is not unreasonable for compression perpendicular to the grain.

Timber-Steel Sill:

$$W = \frac{108.2}{4} = 27.1 \text{ K/ft} \qquad M- = \frac{WL^2}{12} = 27.1 \times 1.33 = 36.2 \text{ K'}$$

$$M+ = \frac{WL^2}{24} = 18.1 \text{ K'}$$

try b = 1.0 ft

for timber $\quad S = 18.1 \times 1.5 \times \frac{12}{6} = 54.3 \quad 2d^2 = 5.43$
$$d^2 = 27.2 \quad d = 5.21$$

use 2 - 6" x 6"

for steel $\quad S = 18.1 \times \frac{12}{50} = 4.35$

use 7 ⊏ 9.8    S = 6.0

2 - 6" x 6"



7⊏?.8

1" dia.
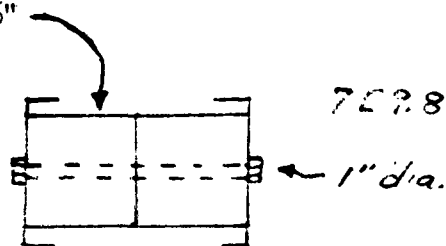
splice timbers and channels between arches

End Wall:
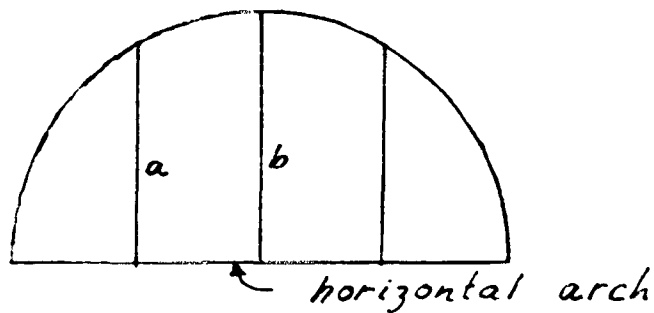


horizontal arch

Vertical beam

    Span = 8'
    W = 13.53 kips/ft

$M = \dfrac{WL^2}{8} = 8 \times 13.53 = 108.2 \ K'$    $S = 108.2 \times \dfrac{12}{50} = 26$

    Use 10 WF 25    S = 26.4

$R_a = 1/2 \left(16 + \dfrac{32}{3}\right) 3380 = 45 \ Kips$

$R_b = 1/2 \times 32 \times 3380 = 54.1 \ Kips$

234

Horizontal Arch:

Analyze one-half of arch
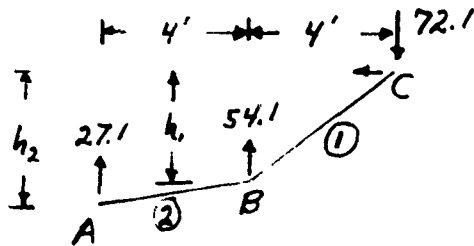


$$L_1 = (h_1^2 + 16)^{1/2}$$

$$L_2 = ([h_2 - h_1]^2 + 16)^{1/2}$$

at Joint C  $\Sigma F_y = 0$

$$F_1 = \frac{(h_1^2 + 16)^{1/2}}{h_1} \quad 72.1 \tag{1}$$

at Joint B  $\Sigma F_x = 0$

$$\frac{F_2 4}{[(h_2-h_1)^2 + 16]^{1/2}} = \frac{F_1 4}{(h_1^2 + 16)^{1/2}} \tag{2}$$

at Joint A  $\Sigma F_y = 0$

$$F_2 = 27.1 \frac{[(h_2 - h_1)^2 + 16]^{1/2}}{h_2 - h_1} \tag{3}$$

from equations (3) and (1), substitute the values of $F_2$ and $F_1$ into equation (2).

$$\frac{27.1}{h_2 - h_1} = \frac{72.1}{h_1} \qquad 72.1\,h_2 = 72.1\,h_1 = 27.1\,h_1$$

$$72.1\,h_2 = 99.2\,h_1$$

$$h_2 = 1.372\,h_1$$

Let $h_1 = 3.0$ ft    then $h_2 = 4.12$ ft

$F_1 = 120.0$ Kips    $F_2 = 100.5$ Kips

Use $F_a = 40,000$ psi    $A = \frac{120}{40} = 3$ in.$^2$

Use a 4 WF 13    $A = 3.82$

## CONCRETE-METAL STRUCTURES

Ultimate Strength Design Method of Shelter.

Design overpressure - 20 psi

Earth pressure assuming 5 ft of cover $(100 \text{ lb/ft}^3$ earth weight)
$= (100 \text{ lb/ft}^3)(5 \text{ ft})/(144 \text{ in.}^2/\text{ft}^2) = 3.5$ psi

Total static design load = 20 + 3.5 = 23.5 psi = 3380 psf

Side Loading:

Cohesionless soil $\qquad p = wh \tan^2 (45° - \frac{\phi}{2})$

Where: $p$ = side pressure in psi
$w$ = unit weight of soil
$h$ = depth of soil
$\phi$ = angle of repose of soil

$P_{13.5} = \frac{100}{144} (13.5) \tan^2 (45° - \frac{26}{2}) = \frac{1350}{144} \tan^2 (32°)$

$P_{13.5} = 9.37 (0.6249)^2 = 3.66$ psi

$P_5 = (3.66) \frac{5}{13.5} = 1.355$

Cohesive soil $\qquad p = wh \tan^2 (45° - \frac{\phi}{2}) - 2c \tan (45° - \frac{\phi}{2})$

Where $c$ = cohesive strength of the soil = 400 psf

$P_{13.5} = \frac{100}{144} (13.5) \tan^2 (45° - \frac{14}{2}) - 2\left(\frac{400}{144}\right) \tan \left(45° - \frac{14}{2}\right)$

$P_{13.5} = \frac{1350}{144} \tan^2 (38°) - \frac{800}{144} \tan (38°)$

$P_{13.5} = 9.37 (0.7813)^2 - 5.56(0.7813) = 5.72 - 4.35 = 1.37$ psi

$P_5 = 5.72 \left(\frac{5}{13.5}\right) - 4.35 = 0$

Assume static side loading = 3.5 psi

Assume dynamic side loading = $\dfrac{20}{2}$ = 10 psi

Total static design side loading = 13.5 psf = 1945 psf

Rackle-Lite lightweight roof slabs will be the material used to cover shelter.

Specifications:

| Max. Span | Thickness | Wt/Sq Ft | Compressive Strength | Ultimate Load |
|-----------|-----------|----------|----------------------|---------------|
| 6' - 3"   | 3"        | 12       | 600 psi              | 240 psf       |
| 8' - 4"   | 4"        | 15       | 600 psi              | 240 psf       |

Calculation of maximum span for simply supported member.

Side load 1944 lb/ft$^2$     (3" thickness)

$$\frac{w_1 L_1^2}{8} = \frac{w_2 L_2^2}{8} \qquad w_1 = 240 \text{ psf} \quad L_1 = 6'\text{-}3" \quad w_2 = 1944 \text{ psf} \quad L_2 = ?$$

$$L_2 = \frac{\sqrt{w_1 L_1^2}}{w_2} = \frac{\sqrt{(240)(6.25)^2}}{1944}$$

$$\underline{L_2 = 2.19' = 2'\text{-}2.28"}$$

(4" thickness)

$$L_2 = \frac{\sqrt{w_1 L_1^2}}{w_2} \qquad w_1 = 240 \text{ psf} \qquad L_1 = 8'\text{-}4" \qquad w_2 = 1944 \text{ psf}$$

$$L_2 = \sqrt{\frac{240(8.33)^2}{1944}}$$

$$\underline{L_2 = 2.92' = 2'\text{-}11.05"}$$

Roof load 3384 lb/ft$^2$     (4" thickness)

$$L_2 = \frac{\sqrt{w_1 L_1^2}}{w_2} \qquad w_1 = 240 \text{ psf} \qquad L_1 = 8'\text{-}4" \qquad w_2 = 3384 \text{ lb/ft}^2$$

$$L_2 = \sqrt{\frac{240(8.33)^2}{3384}}$$

$$\underline{L_2 = 2.34' = 2'\text{-}4.08"}$$

## MAIN SHELTER

Design of Steel Frame for Shelter. 2' Intervals.

By using the ultimate design method, the design stress will be 50,000 psi in bending and direct compression.

The loading on and the dimensions of the steel frame are shown at right. The connections at b and c are rigid and at a and d are hinged.
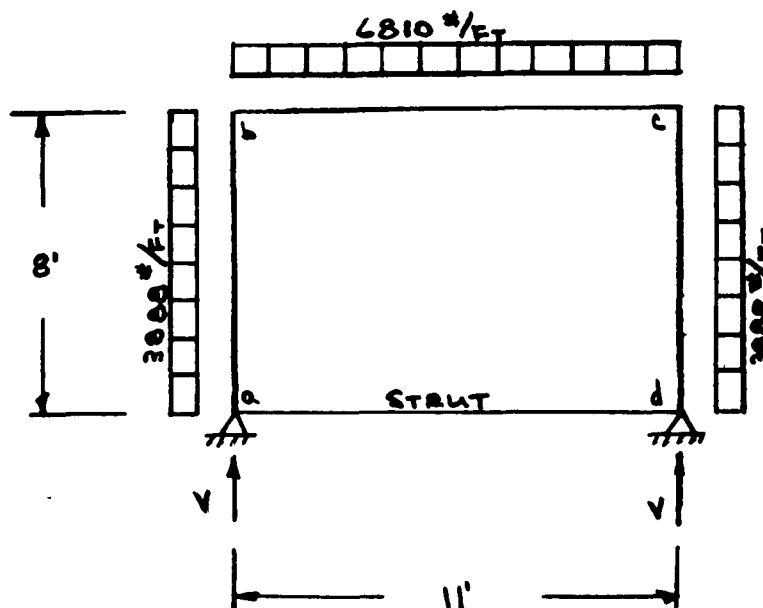


Loading on Beam

Static pressure load $= (2 \text{ ft}) \left(144 \frac{\text{in.}^2}{\text{ft}^2}\right) (23.5 \text{ psi}) = 6,760 \text{ lb/ft}$

Assumed Beam Weight $= 20 \text{ lb/ft}$

Precast concrete roof $= 30 \text{ lb/ft}$
Rackle-Lite 4" x 24" x 6' (15 psf)

Total $6,810 \text{ lb/ft}$

Loading on Column

Static pressure load $= (13.5 \text{ psi})(2 \text{ ft}) \left(144 \frac{\text{in.}^2}{\text{ft}^2}\right) = 3,888 \text{ lb/ft}$

$\Sigma F_V = 2V - (6810)(11) = 0$

$V = \frac{1}{2}(74,910)$

$\underline{V = 37,455 \text{ lb}}$

To determine the acting moments at joints b and c, the moment distribution method will be used.

The solution for the moments will be solved assuming that the moment of inertias for the members are equal. If members are used where the I's are not the same, the moment distribution method will be used again with actual I's solving for the actual moment. This moment will then be used to determine if the size of the existing member must be changed.

| ⑥ | .647 | .353 | ⑪ | .353 | .647 | ⑥ |
|---|---|---|---|---|---|---|
| +20.8 | -20.8 | +68.6 | | -68.6 | +20.8 | -20.8 |
| -20.8 | -10.4 | -13.2 | | +13.2 | -10.4 | +20.8 |
| | -24.2 | 6.6 | | - 6.6 | +24.2 | |
| | - 4.3 | - 2.3 | | + 2.3 | + 4.3 | |
| | - 0.75 | 1.15 | | - 1.15 | + 0.75 | |
| | - 0.13 | - 0.41 | | + 0.41 | + 0.13 | |
| | - 0.02 | 0.20 | | - 0.20 | + 0.02 | |
| | | - 0.07 | | + 0.07 | | |
| | | 0.04 | | - 0.04 | | |
| | | - 0.01 | | + 0.01 | | |
| 0.00 | -60.60 | +60.60 | | -60.60 | +60.60 | 0.00 |

Design moment will be 60,600 ft-lb = $M_d$

Calculation of unknowns on beam ab and dc



$$\overset{+}{\curvearrowright}\ \Sigma M_0 = M_d + 8H - 1/2(8)^2(3888) = 0$$

$$H = \frac{1}{8}[124,416 - 60,600]$$

$$\underline{H = 7,977\ lb}$$

$$\Sigma F = H + H_1 - 8(3888) = 0$$

$$H_1 = 31,104 - 7,977$$

$$H_1 = 23,127\ lb$$

$$\underline{M_{max.} = M_d = 60,600\ ft\text{-}lb}$$



SHEAR
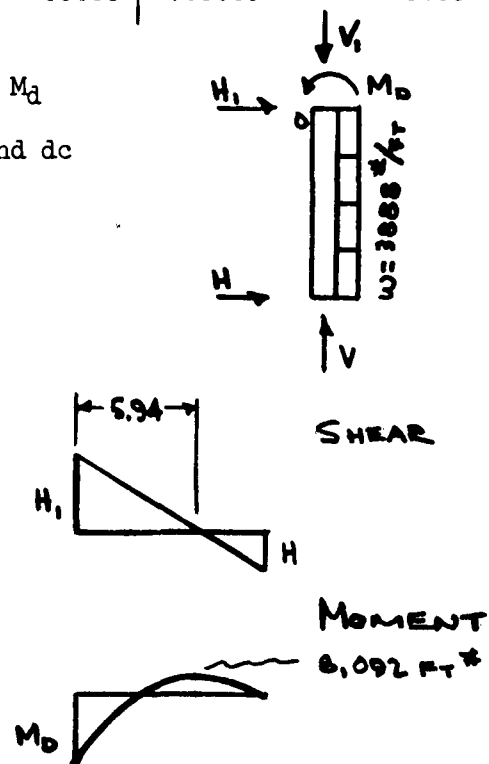
MOMENT
6,092 FT#

Design of Beam bc

Maximum bending moment $\qquad$ $M_d = 60,600$ ft-lb $= 727,200$ in.-lb

Axial compressive load $\qquad$ $H_1 = 23,127$ lb

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1 \qquad f_a = \frac{H_1<}{A} \quad f_b = \frac{M_d}{S_{xx}}$$

$$F_a = F_b = 50,000 \text{ psi}$$

$$\frac{1}{50,000}\left[\frac{23,127}{A} + \frac{727,200}{S_{xx}}\right] \leq 1$$

$$\frac{.462}{A} + \frac{14.544}{S_{xx}} \leq 1$$

Try 8 WF 20 $\quad A = 5.88$ in.$^2$ $\quad S_{xx} = 17.9$ in.$^3$ $\quad I_{xx} = 69.2$ in.$^4$

$$\frac{.462}{5.88} + \frac{14.544}{17.0} = .079 + .855 = .934 < 1$$

Use 8 WF 20

Design of Beams ab and dc

Maximum bending moment $\qquad$ $M_{max.} = 60,600$ ft-lb $= 727,200$ in.-lb

Axial compressive load $\qquad$ $V = 37,455$ lb

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

$$\frac{1}{50,000}\left[\frac{37,455}{A} + \frac{727,000}{S_{xx}}\right] \leq 1$$

$$\frac{.7491}{A} + \frac{14.544}{S_{xx}} \leq 1$$

Try 8 WF 20 $\quad A = 5.88$ in.$^2$ $\quad S_{xx} = 17.0$ in.$^3$ $\quad I_{xx} = 69.2$ in.$^4$

$$\frac{.7491}{5.88} + \frac{14.544}{17.0} = .128 + .855 = .983 < 1$$

Use 8 WF 20

Design of Strut

Axial compressive load $H = 7,977$ lb

$$A = \frac{H}{S} = \frac{7977}{50,000} = .15954 \text{ in.}^2$$

Use L 1 x 1 x 1/8    $A = .23$ in.$^2$    .80 lb/ft

Design of enclosed bulkhead

### Foundation

### Sides

All loads 37,470 lb



$$L = 28 \text{ at } 2' = 56$$
$$2 \text{ at } 7' = \underline{14}$$
$$7 \text{ at } 6' = \overline{42}$$

$$M = \frac{wL}{12} = \frac{37,470 \times 24}{12} = 74,940 \text{ in.-lb}$$

$$M = \frac{wL}{24} = 37,470 \text{ in.-lb}$$
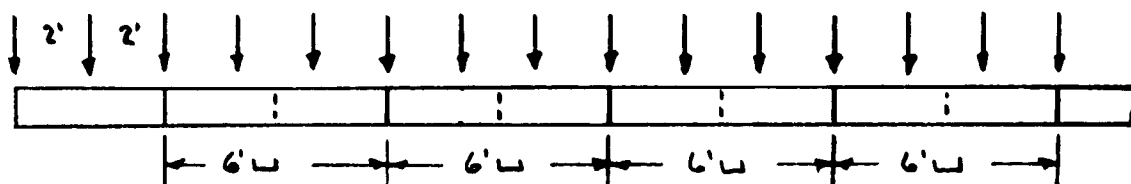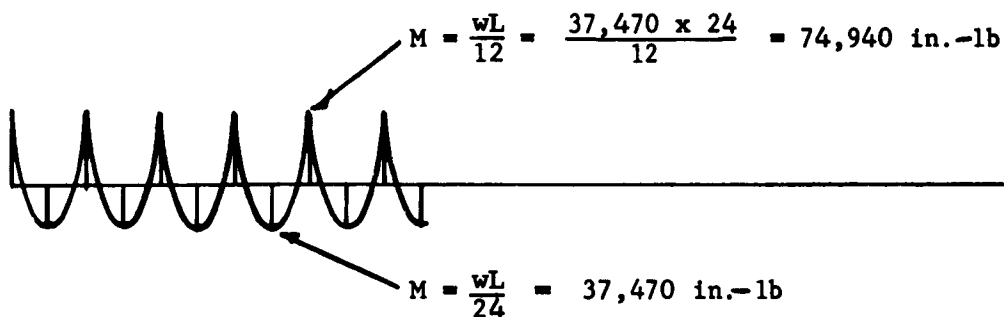
Assume I is constant throughout
Sills spliced by channels

Max. M in $\mathsf{C}$ S = 37,470 in.-lb

$$f_s = 50,000 \qquad S = \frac{M}{f} = \frac{37,470}{50,000} = 0.75$$

Max. M in sill = 74,940 in.-lb

$$f^2 = f_s \times \frac{E_t}{E_s} \times \frac{3}{4} = 50,000 \cdot x \frac{1600}{30,000} \times \frac{3}{4} = 2000 \text{ psi}$$

$$S = \frac{M}{f} = \frac{74,940}{2,000} = 37.5 = \frac{bh^2}{6} \qquad \text{Let } b = 11.5"$$

$$h^2 = \frac{6 \times 37.5}{11.5} = 19.6 \qquad h_{min.} = 4.44" \qquad \text{Try } 5" \times 12"$$
$$S_t = 38.8$$

Try 6 ⊔ 8.2     S = 4.3 x 2 = 8.6
$M_{rc}$ = Sf = 8.6 ⨉ 50,000 = 430,000 in.-lb > 37,470
$M_{rt}$ = 38.8 x 50,000 x $\frac{1600}{30,000}$ x $\frac{2.25}{3}$ = 78,000 in.-lb

Timber stress at splices

$M_t$ = 74,940 in.-lb     $S_t$ = 99.2

$$f = \frac{M_t}{S_t} = \frac{74,940}{38.8} = 1930 \text{ psi} < 6000$$

Try new h, minimum ⊏ s, 8 WF 20 columns

$$S_t = \frac{M}{f} = \frac{74,940}{6000} = 12.5$$

Min. ⊏ = 3 ⊔ 4.1     T = 1-3/4"     a = 1-1/4"

b min. = 13 + 1-1/4 x 2 = 15.5", say 16.0" nom.

$$s = \frac{bh^2}{6} = 12.5 \qquad h^2 = \frac{12.5 \times 6}{15.5} = 4.83$$

$h_{min.}$ = 2.20" > 1-3/4"     Try wider channel

Try 4 ⊔ 5.4     T = 2-3/4"     a = 1-3/8"

$b_{min.}$ = 10 + 1-3/8 x 2 = 10 + 2-3/4 = 12-3/4"     Try 14"

$$h^2 = \frac{12.5 \times 6}{13.5} = 5.55 \qquad h_{min.} = 2.46" \qquad \text{Try } 14" \times 3"$$

Shear     $H = \frac{3V}{2bh}$
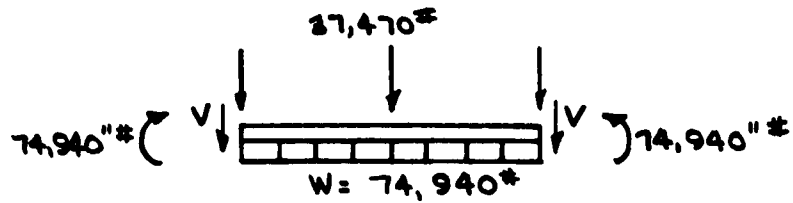
$$\Sigma F_v = 0 \quad 3 \times 37{,}470 + 2V = 74{,}940 \qquad 112{,}410 + 2V = 74{,}940$$

$$2V = -37{,}470 \qquad V = -18{,}735 \text{ lb}$$

$$H = \frac{3 \times 18{,}735}{2 \times 13.5 \times 2.625} = 795 \text{ psi} > 640 \text{ allow.}$$

Use 4 �└ 5.4, 14" x 3" timbers

Bearing Plates
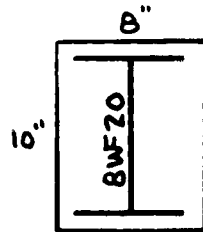


Weld to columns

$$P = 37{,}470 \qquad C_1 = 1100 \text{ psi}$$

$$A_{min.} = \frac{37{,}470}{1100} = 34 \text{ sq in.}$$

$$C = \frac{37{,}470}{13 \times 6} = 481 \text{ psi}$$

$$M = \frac{wL^2}{2} = \frac{481 \times 1}{2} = 240 \text{ in.-lb}$$

$$S = \frac{M}{f} = \frac{240}{60{,}000} = .0040 = \frac{bt^2}{6}$$

$$t^2 = \frac{6 \times .0040}{1} = 0.024$$

$$t = 0.155 \text{ in.} \qquad \text{Use } 1/4" \text{ PL}$$

Use 10" x 8" x 1/4" plates
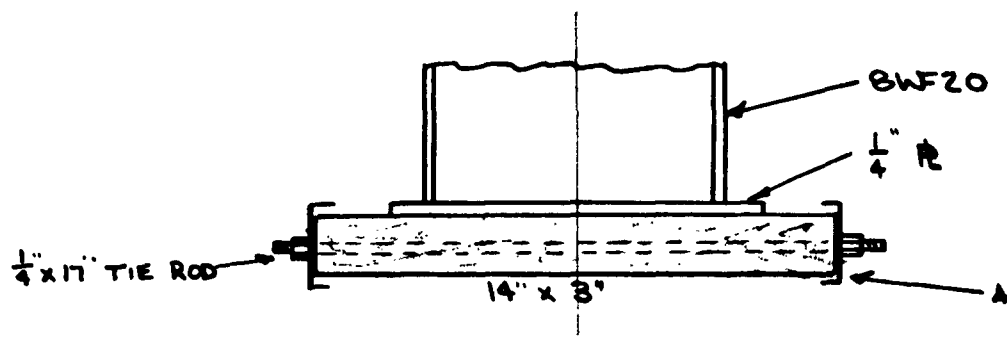
Soil bearing

$$P = \frac{37,470}{2 \times \frac{13.5}{12}} = 16,840 \text{ psf} = 8.4 \text{ Tsf}$$

## Design of Enclosed Bulkhead
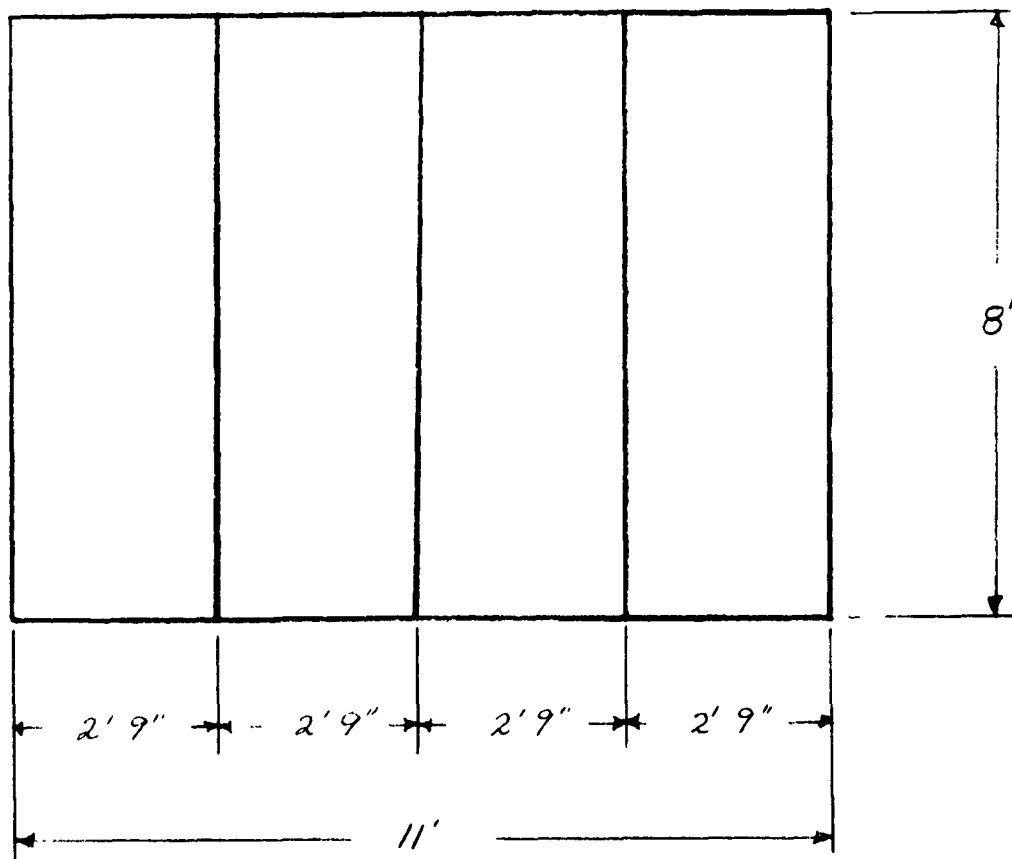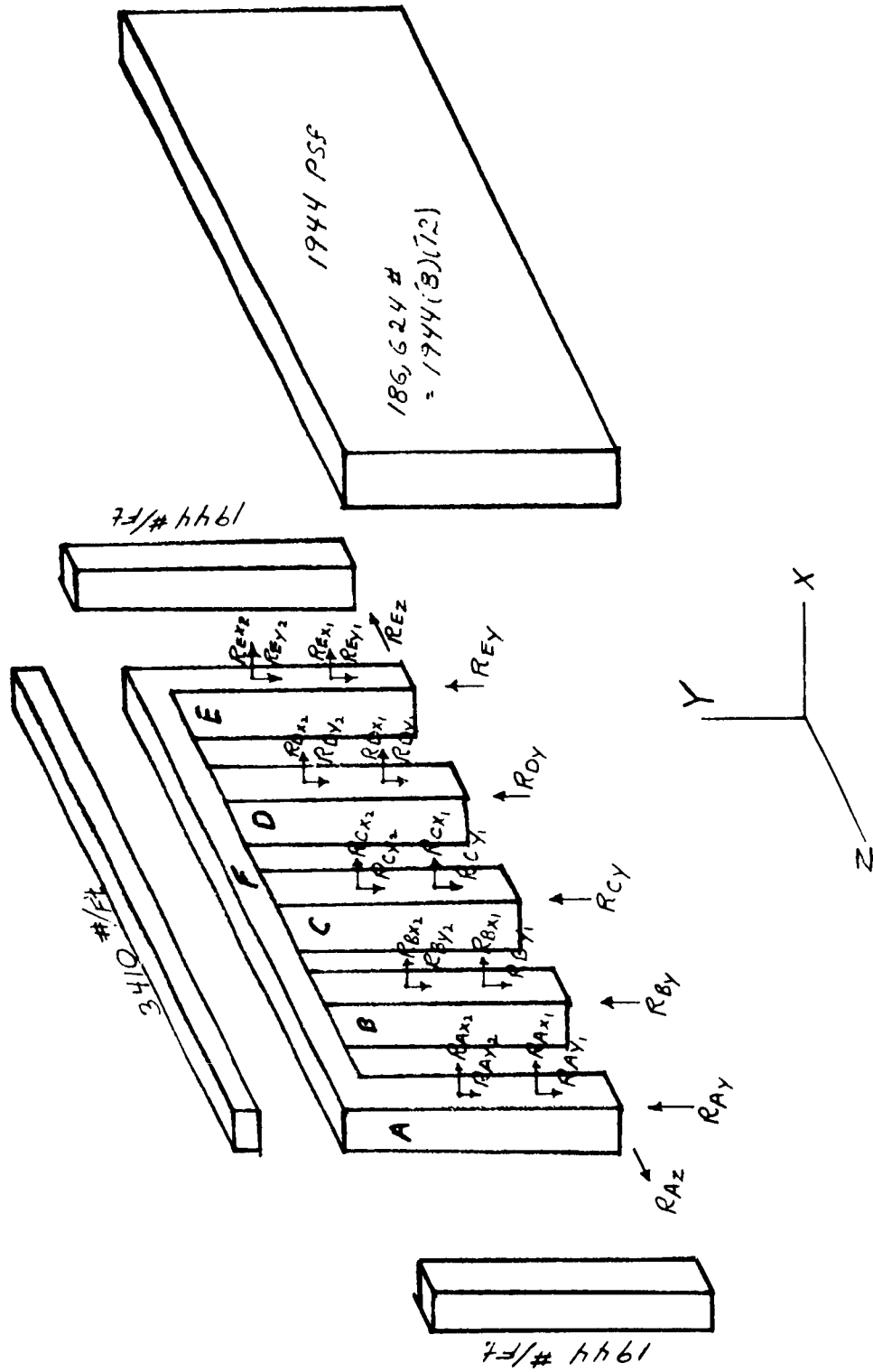
Bulkhead Loading

Top Load

$$\text{Static pressure load} = (1\text{ ft})(23.5\text{ psi})(144\ \frac{\text{in.}^2}{\text{ft}^2}) = 3384\text{ lb/ft}$$

Assumed beam weight  = 11 lb/ft

Precast concrete = (15 psf)(1 ft)  = 15 lb/ft

3410 lb/ft

Side Load

$$\text{Static pressure load} = (1\text{ ft})(13.5\text{ psi})\left(144\ \frac{\text{in.}^2}{\text{ft}^2}\right) = 1944\text{ lb/ft}$$

End Load

$$\text{Static pressure load} = (13.5\text{ psi})\left(144\ \frac{\text{in.}^2}{\text{ft}^2}\right) = 1944\text{ psf}$$

Assume frame loading on end as shown below. Assume no end loading on Beam F.

Beam A = 23,328 lb    2916 lb/ft
Beam E = 23,328 lb    2916 lb/ft
Beam B = 46,656 lb    5832 lb/ft
Beam C = 46,656 lb    5832 lb/ft
Beam D = 46,656 lb    5832 lb/ft
186,624 lb
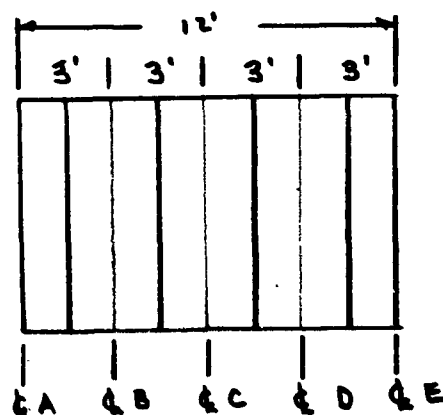


The loading on each bar in the X direction is shown below.

$$R_{ax_2} = R_{ax_1} = 11,664\text{ lb}$$

$$R_{ex_2} = R_{ex_1} = 11,664\text{ lb}$$

$$R_{bx_2} = R_{bx_1} = 23,328\text{ lb}$$

$$R_{cx_2} = R_{cx_1} = 23,328\text{ lb}$$

$$R_{dx_2} = R_{dx_1} = 23,328\text{ lb}$$
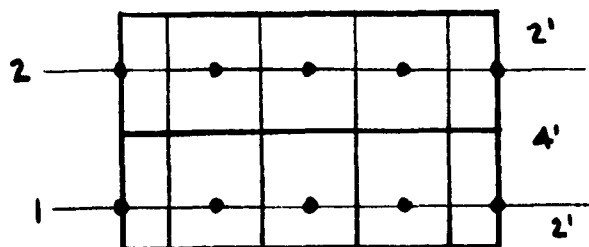
The approximate dimensions
for the placement of the
deadman are shown at right.

$R_{ay_2} = R_{ey_2} = 11,664$ lb

$R_{by_2} = R_{cy_2} = R_{dy_2} = 23,328$ lb

$R_{ay_1} = R_{ey_1} = R_{ax_1}/\tan\phi = 5,000$ lb

$R_{by_1} = R_{cy_1} = R_{dy_1} = R_{bx_1}/\tan\phi =$

     10,000 lb
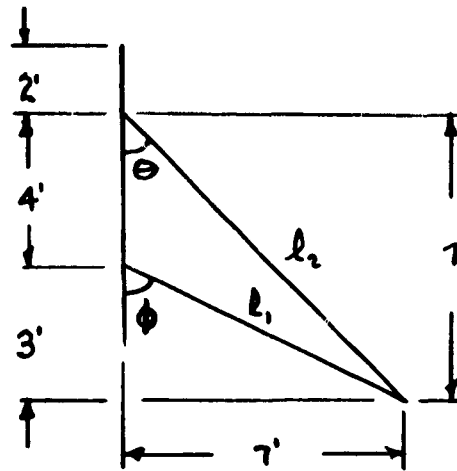


$\theta = 45°$      $L_2 = 9.9'$

$\phi = 66.8°$      $L_1 = 7.63'$

Design of tie rods from end bulkhead to deadman.

    The components of the loads acting on the rods were found above.
The resultants of these components will be the loads acting on the
tie rods which constitute the design loads.

$$R_{e_1} = R_{a_1} = \sqrt{(R_{ax_1})^2 + (R_{ay_1})^2} = [(11,664)^2 + (5,000)^2]^{1/2} = 12,680 \text{ lb}$$

$$R_{e_2} = R_{a_2} = \sqrt{(R_{ax_2})^2 + (R_{ay_2})^2} = [(11,664)^2 + (11,664)^2]^{1/2} = 16,480 \text{ lb}$$

$$R_{d_1} = R_{c_1} = R_{b_1} = \sqrt{(R_{bx_1})^2 + (R_{by_1})^2} = [(23,328)^2+(10,000)^2]^{1/2} = 25,350 \text{ lb}$$

$$R_{d_2} = R_{c_2} = R_{b_2} = \sqrt{(R_{bx_2})^2 + (R_{by_2})^2} = [(23,328)^2+(23,328)^2]^{1/2} = 32,980 \text{ lb}$$

Design of rods $A_1$ and $E_1$

$$A = \frac{P}{F_a} = \frac{12,680}{50,000} = .254 \text{ in.}^2 \qquad \underline{\text{Bar } \tfrac{5}{8} \phi}$$

Design of rods $A_2$ and $E_2$

$$A = \frac{P}{F_a} = \frac{16,480}{50,000} = .3296 \text{ in.}^2 \qquad \underline{\text{Bar } \tfrac{11}{16} \phi}$$

Design of Rods $B_1$ $C_1$ and $D_1$

$$A = \frac{P}{F_a} = \frac{25,350}{50,000} = .5070 \text{ in.}^2 \qquad \text{Bar } \frac{13}{16} \phi$$

Design of rods $B_2$ $C_2$ and $D_2$

$$A = \frac{P}{F_a} = \frac{32,980}{50,000} = .6596 \text{ in.}^2 \qquad \text{Bar } \frac{15}{16} \phi$$

Free Body Diagrams of End Bulkhead Members

BEAM A
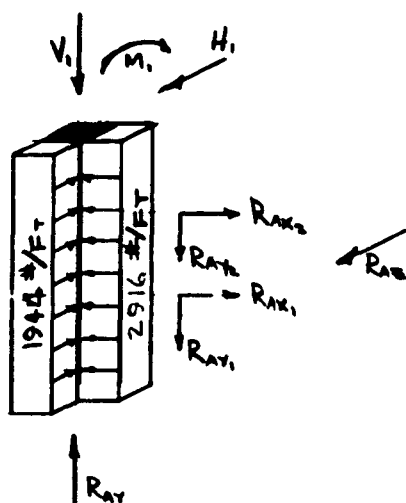
BEAM B



BEAM F

Calculation of Moments on Enclosed Bulkhead by Moment Distribution Method

| | A | B | | C | | D | | E | | F | | G |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 8' | 3' | | 3' | | 3' | | 3' | | 8' | |
| | | −12.994 | | | | | | | | +12.952 | | |
| DF | 0 | .58 | .42 | .50 | .50 | .50 | .50 | .50 | .50 | .42 | .58 | 0 |
| | +10.368 | −10.368 | +2.558 | −2.558 | +2.558 | −2.558 | +2.558 | −2.558 | +2.558 | −2.558 | +10.368 | −10.368 |
| | −10.368 | − 5.184 | | +2.729 | | | | | | | + 5.184 | +10.368 |
| | | + 7.536 | +5.458 | −1.364 | −1.364 | − .682 | | | | − .042 | | |
| | | − .682 | | | + .170 | + .341 | + .170 | − .085 | | −5.440 | −7.512 | |
| | | | | | | − .042 | − .085 | −2.720 | | + .680 | | |
| | | | | − .159 | − .319 | + .680 | +1.360 | +1.360 | | | | |
| | | | | − .005 | − .002 | − .319 | − .159 | | | | | |
| | | + .397 | | − .071 | − .035 | | | | | | | |
| | | | | + .009 | + .018 | + .018 | + .009 | + .009 | + .075 | + .037 | | |
| | | | | | | + .037 | + .075 | + .075 | − .150 | − .301 | − .416 | |
| | | − .002 | − .005 | − .005 | − .037 | + .037 | − .018 | − .018 | + .075 | + .037 | | |
| | | + .287 | + .143 | + .143 | + .002 | + .002 | | | | | | |
| | | − .035 | − .071 | − .071 | | − .001 | | | | | | |
| | | | | − .018 | + .018 | + .004 | + .009 | + .009 | + .009 | + .004 | | |
| | | | | + .004 | + .004 | | − .008 | − .008 | | − .017 | − .024 | |
| | +.019 | + .002 | + .004 | + .004 | + .002 | + .002 | + .004 | + .004 | + .004 | + .002 | | |
| | | + .014 | + .007 | − .003 | − .003 | − .003 | − .001 | − .001 | | − .001 | − .001 | |
| | | − .001 | − .003 | − .003 | | | | | | | | |
| | | | | − .001 | | | | | | | | |
| | 0 | −7.600 | +7.599 | −1.118 | +1.120 | −3.276 | +3.276 | −1.119 | +1.118 | −7.599 | +7.599 | 0 |

Ratio of $\dfrac{I_{ab}}{I_{bc}} \approx 5$

$$M = \frac{(1944)(8)^2}{12} = 10,368$$

$$M = \frac{(1944)(8)^2}{12} = 10,368$$

$$M = \frac{3410(3)^2}{12} = 2557.5$$

$$K_{ab} = E \frac{(5)}{(8)}\left(\frac{3}{4}\right) = .468$$

$$\frac{.468}{.801} = .585$$

$$K_{bc} = E \frac{(1)}{3} = .333$$

$$\frac{.333}{.801} = .415$$

Calculation of unknowns on Beam F (use superposition)
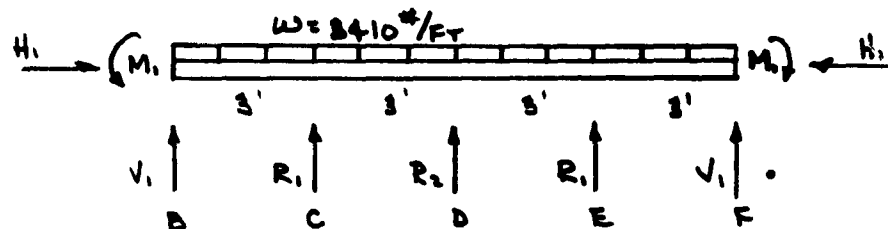


Section B-C



$V_B = V_C = \frac{1}{2}(3)(3410)$     $R_B = R_C = \frac{1}{3}(M_B + M_C)$     $V_1 = V_B + R_B = 5115$
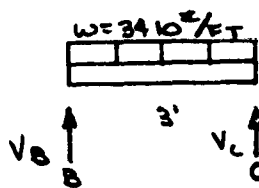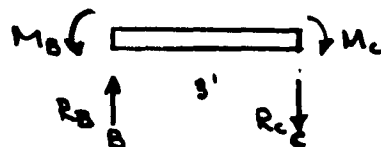
$V_B = V_C = 5115$ lb     $R_B = R_C = \frac{1}{3}(7,600 - 1,120)$       $+ 2160$

                       $R_B = R_C = 2161$ lb     $\underline{V_1 = 7275 \text{ lb}}$

Section C-D



$V_C' = V_D = \frac{1}{2}(3)(3410)$     $R_C' = R_D = \frac{1}{3}(M_C + M_D)$     $R_1 = V_C - R_C + V_C'$
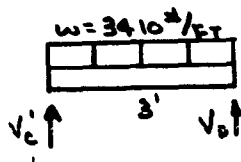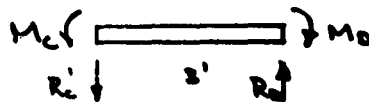
$V_C' = V_D = 5115$ lb     $R_C' = R_D = \frac{1}{3}(1,120 - 3,276)$       $- R_C'$

                       $R_C' = R_D = 719$ lb     $R_1 = 5115 - 2161 +$
                                              $5115 - 719$

                                        $\underline{R_1 = 7,350 \text{ lb}}$

250

## Section D-E
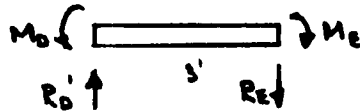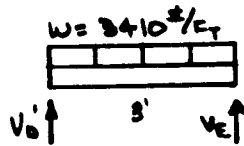


$$V_D' = V_E = \tfrac{1}{2}(3)(3410)$$

$$V_D' = V_E = 5115 \text{ lb}$$

$$R_D' = R_E = \tfrac{1}{3}(M_D + M_E)$$

$$R_D' = R_E = \tfrac{1}{3}(3{,}276 - 1{,}120)$$

$$R_D' = R_E = 719 \text{ lb}$$

$$R_2 = V_D + R_D +$$

$$V_D' + R_D'$$

$$R_2 = 5115 + 719$$
$$+ 5115 + 719$$

$$\underline{R_2 = 11{,}668 \text{ lb}}$$

## Calculation of unknowns on Beam A in Y-Z plane

$$\Sigma F_Y = V_1 + R_{AY_1} + R_{AY_2} - R_{AY} = 0$$

$$R_{AY} = 7275 + 5000 + 11664$$

$$\underline{R_{AY} = 23{,}939}$$

$$\Sigma M_O = 0 = \tfrac{1}{2}(1944)(8)^2 - R_{az}8 - M_1$$

$$R_{az} = \tfrac{1}{8}[62{,}208 - 7{,}600]$$

$$\underline{R_{az} = 6826}$$

$$\Sigma F_z = 0 = R_{az} + H_1 - 8(1944)$$

$$H_1 = 15{,}552 - 6826$$

$$\underline{H_1 = 8726}$$

$$\underline{M_{max.} = 11{,}990 \text{ ft-lb}}$$
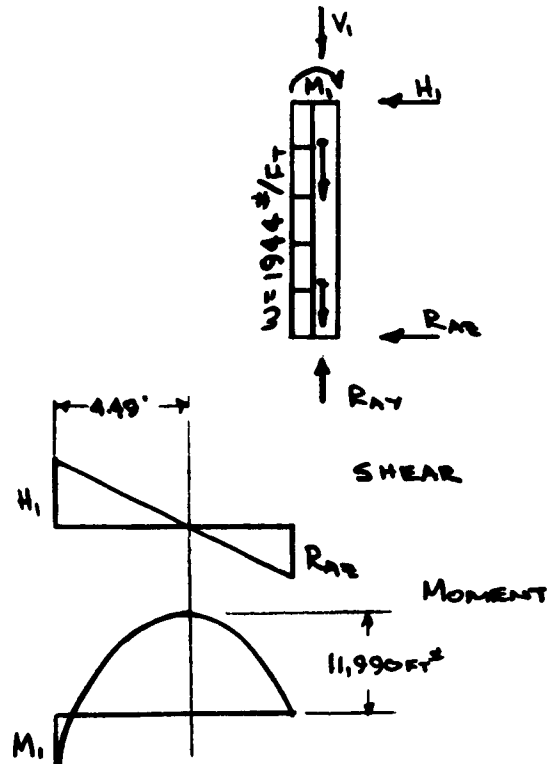
Calculation of unknowns on Beam A in the X-Y plane

To design beam, the max.
bending moment must be
calculated. This is done
by the shear and moment
diagrams drawn at right.
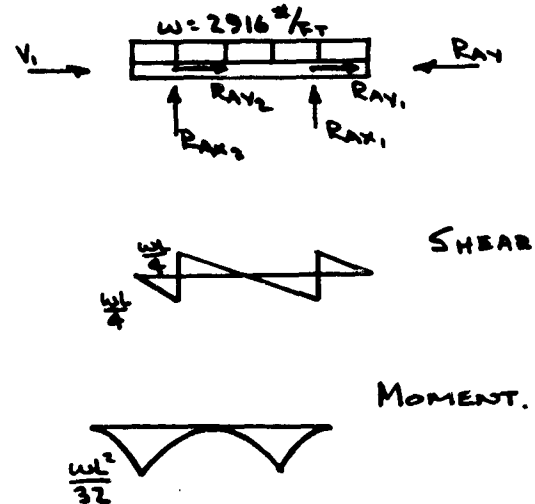
$$M_{max.} = \frac{wL^2}{32} = \frac{(2916)(64)}{32}$$

$$\underline{M_{max.} = 5,832 \text{ ft-lb}}$$

Calculations of unknowns on Beam B in the X-Y plane

$$\Sigma F_y = R_1 + R_{by_2} + R_{by_1} - R_{by} = 0$$

$$R_{by} = 7350 + 23,328 + 10,000$$

$$\underline{R_{by} = 40,678 \text{ lb}}$$

$$M_{max.} = \frac{wL^2}{32} = \frac{(5832)(8)^2}{32}$$
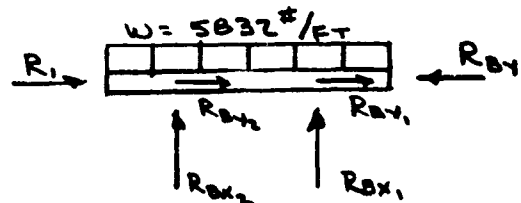
$$\underline{M_{max.} = 11,664 \text{ ft-lb}}$$

Calculation of unknowns on Beam C in the X-Y plane

$$\Sigma F_y = R_2 + R_{cy_2} + R_{cy_1} - R_{cy} = 0$$

$$R_{cy} = 11,668 + 23,328 + 10,000 \qquad \text{Same figure as above}$$

$$\underline{R_{cy} = 44,996}$$

$$M_{max.} = \frac{wL^2}{32} = \frac{(5832)(8)^2}{32}$$

$$\underline{M_{max.} = 11,664 \text{ ft-lb}}$$

Design of Beam F

Maximum bending moment $\qquad$ $M_1 = 7600$ ft-lb

Axial compressive load $\qquad$ $H_1 = 8726$ lb

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

$$\frac{1}{50,000}\left[\frac{8726}{A} + \frac{7600(12)}{S_{yy}}\right] \leq 1$$

$$\frac{.1745}{A} + \frac{1.824}{S_{yy}} \leq 1$$

Try L 5 x 3-1/2 x 5/8    $A = 3.03$ in.$^2$    $S_{yy} = 1.9$ in.$^3$    $I_{yy} = 4.8$ in.$^4$

$$\frac{.1745}{4.92} + \frac{1.824}{1.9} = .036 + .960 = .996 < 1$$

Try L 5 x 5 x 5/16    $A = 3.03$ in.$^2$    $S_{xx} = 2.0$ in.$^3$    $I_{xx} = 7.4$ in.$^4$

$$\frac{.1745}{3.03} + \frac{1.824}{2.0} = .058 + .912 = .970 < 1$$

Use L 5 x 3-1/2 x 5/8

Design of Beam A

Loading in X-Y plane and Y-Z plane

Maximum bending moment in X-Y plane $\qquad$ = 5,832 ft-lb

Maximum bending moment in Y-Z plane $\qquad$ = 11,990 ft-lb

Axial compressive load $\qquad$ = 23,939 lb

$$\frac{f_a}{F_a} + \left(\frac{f_b}{F_b}\right)_{xx} + \left(\frac{f_b}{F_b}\right)_{yy} \leq 1$$

$$\frac{1}{50,000}\left[\frac{23,939}{A} + \frac{11,990(12)}{S_{xx}} + \frac{5832(12)}{S_{yy}}\right] \leq 1$$

$$\frac{.4747}{A} + \frac{2.878}{S_{xx}} + \frac{1.3997}{S_{yy}} \leq 1$$

Try 5 WF 16    $A = 4.70$ in.$^2$    $S_{xx} = 8.53$ in.$^2$    $S_{yy} = 3.00$ in.$^3$

$I_{xx} = 21.3$ in.$^4$

$$\frac{.4747}{4.70} + \frac{2.878}{8.53} + \frac{1.3997}{3.00} = .101 + .337 + .466 = .904 < 1$$

Ratio of $\dfrac{I_{ab}}{I_{bf}} = \dfrac{21.3}{4.8} = 4.46 \approx 5$

$$\frac{I_{ab}}{I_{bf}} = \frac{21.3}{7.4} = 2.88 \approx 3$$

Since ratio of 5 was used in designing beams in the moment distribution method, use 5 WF 16 and L 5 x 3-1/2 x 5/8
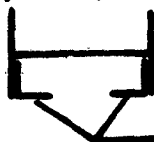
Use 5 WF 16     Use L 1 x 1 x 3/16 to sustain bearing end load

Design of Beam B, D

Loading in X-Y plane only

| | |
|---|---|
| Maximum bending moment | $M_{max.} = 11,664$ ft-lb |
| Axial compressive load | $R_{by} = 40,678$ lb |

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1$$

$$\frac{1}{50,000}\left[\frac{40,678}{A} + \frac{11664(12)}{S_{xx}}\right] \le 1$$

$$\frac{.8136}{A} + \frac{2.79936}{S_{xx}} \le 1$$

Try 5 I 10     $A = 2.87$ in.$^2$     $S_{xx} = 4.8$ in.$^3$

$$\frac{.8136}{2.87} + \frac{2.79936}{4.8} = .283 + .583 = .866 < 1$$

Use 5 I 10

Design of Beam C

| | |
|---|---|
| Maximum bending moment | $M_{max.} = 11,664$ ft-lb |
| Axial compressive load | $R_{by} = 44,296$ lb |

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

$$\frac{1}{50,000}\left[\frac{44,996}{A} + \frac{11,664(12)}{S_{xx}}\right] \leq 1$$

$$\frac{.8995}{A} + \frac{2.799}{S_{xx}} \leq 1$$

Try 5 I 10    $A = 2.87$ in.$^2$     $S_{xx} = 4.8$ in.$^3$

$$\frac{.8995}{2.87} + \frac{2.799}{4.8} = .314 + .583 = .897 < 1$$

Use 5 I 10

Design of Strut         Axial Load     $R_{az} = 7076$ lb

$$A = \frac{P}{A} = \frac{7076}{50,000}$$

$A = .1417$ in.$^2$

Use L 1 x 1 x 1/8       $A = .23$ in.$^2$       .80 lb/ft

Dimensions of Enclosed Bulkhead
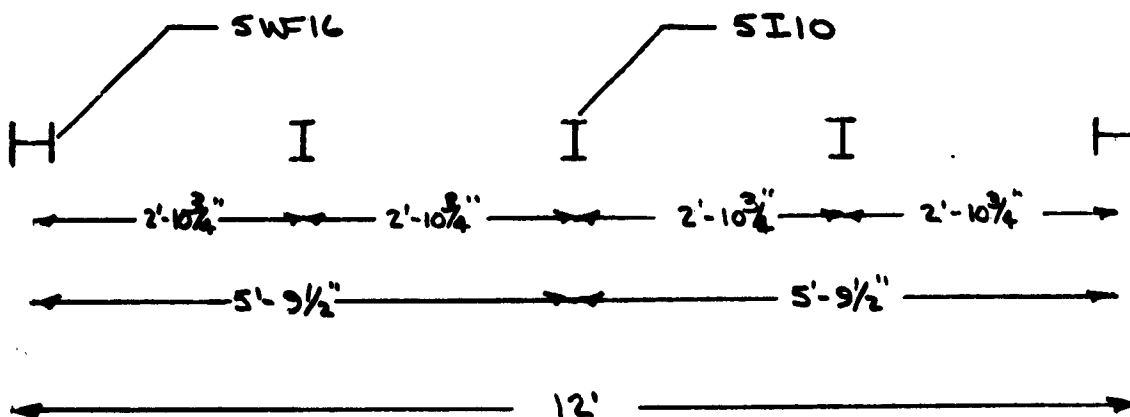
## Closed End
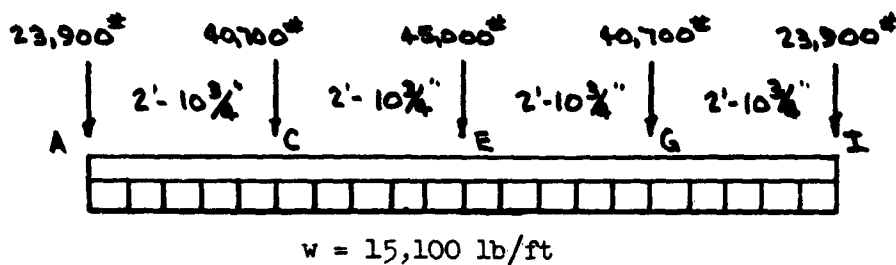


$$w = 15{,}100 \text{ lb/ft}$$

$$w = \frac{23{,}900 \times 2 + 40{,}700 \times 2 + 45{,}000}{11.58} = \frac{47{,}800 + 81{,}400 + 45{,}000}{11.58}$$

$$w = \frac{174{,}200}{11.58} = 15{,}100 \text{ lb/ft}$$

### Shear Diagram
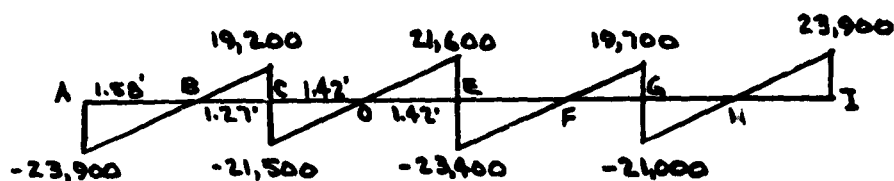


$$\text{Max. Moment} = M_d = 1/2 \times 23{,}900 \times 1.58 + \frac{1}{2} \times 21{,}500 \times 1.42 - \frac{1}{2} \times 19{,}200 \times 1.27$$

$$M_d = 18{,}900 + 15{,}300 - 12{,}200 = 22{,}000 \text{ ft-lb} = 264{,}000 \text{ in.-lb}$$

$$S = \frac{M}{f} = \frac{264{,}000}{6000} = 44 = \frac{bh^2}{6} \qquad \text{Let } b = 16''$$

$$h^2 = \frac{44 \times 6}{15.5} = 17.0 \qquad h = 4.13'' \qquad \text{Try } 16'' \times 5''$$

$$H = \frac{3V}{2bh} = \frac{3 \times 21{,}600}{2 \times 15.5 \times 4.5} = 465 < 640$$

$$B_{soil} = \frac{15{,}100}{\frac{15.5}{12}} = 11{,}700 \text{ psf}$$

### Use 16" x 5"

Bearing plates

$$\underline{5 \text{ WF } 16 - 6" \text{ x } 6" \text{ x } 1/4" \text{ PL}}$$

$$\underline{5 \text{ I } 10 - 7" \text{ x } 6" \text{ x } 1/4"}$$

$A_{E_{min.}} = \dfrac{45,000}{1100} = 41$ sq in.

$A_{A-I_{min.}} = \dfrac{23,900}{1100} = 21.7$ sq in.

Dimensions of Entrance Bulkhead (approximate, for design purposes)

### Y-Z Plane
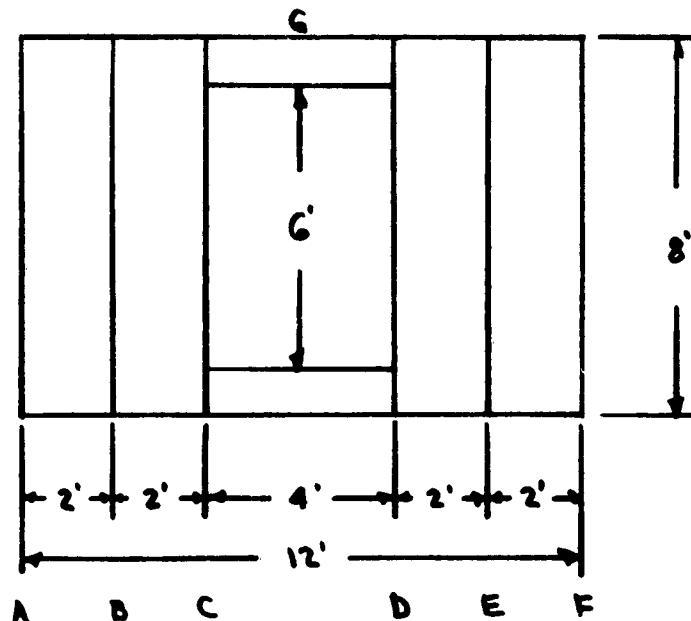


Loading on members in Y-Z plane
    (Assume no end loading on Beam G)

| | | |
|---|---|---|
| Beam A | = 1' (1944 psf) = 1944 lb/ft | 15,552 lb |
| Beam F | = 1' (1944 psf) = 1944 lb/ft | 15,552 lb |
| Beam B | = 2' (1944 psf) = 3888 lb/ft | 31,104 lb |
| Beam E | = 2' (1944 psf) = 3888 lb/ft | 31,104 lb |
| Beam C$_{6'}$ | = 1' (1944 psf) = 1944 lb/ft | 11,664 lb |

Beam $D_{6}$, = 1' (1944 psf) = 1944 lb/ft     11,664 lb
Beam $C_{2}$, = 3' (1944 psf) = 5832 lb/ft     11,664 lb
Beam $D_{2}$, = 3' (1944 psf) = 5832 lb/ft     11,664 lb

Loading in X direction on each bar

$$R_{ax_1} = R_{ax_2} = 7,776 \text{ lb}$$

$$R_{bx_1} = R_{bx_2} = 15,552 \text{ lb}$$

$$R_{cx_1} = R_{cx_2} = 11,664 \text{ lb}$$

$$R_{dx_1} = R_{dx_2} = 11,664 \text{ lb}$$

$$R_{ex_1} = R_{ex_2} = 15,552 \text{ lb}$$

$$R_{fx_1} = R_{fx_2} = 7,776 \text{ lb}$$

The approximate dimensions for the placement of the deadman are shown below.

$$R_{ay_2} = R_{fy_2} = 7,776 \text{ lb}$$

$$R_{by_2} = R_{ey_2} = 15,552 \text{ lb}$$

$$R_{cy_2} = R_{dy_2} = R_{cx_2}/\tan = 12,400 \text{ lb}$$

$$R_{ay_1} = R_{fy_1} = R_{ax_1}/\tan\Theta = 3,890 \text{ lb}$$

$$R_{by_1} = R_{ey_1} = R_{bx_1}/\tan\Theta = 7,780 \text{ lb}$$

$$R_{cy_1} = R_{dy_1} = R_{cx_1}/\tan = 5,100 \text{ lb}$$
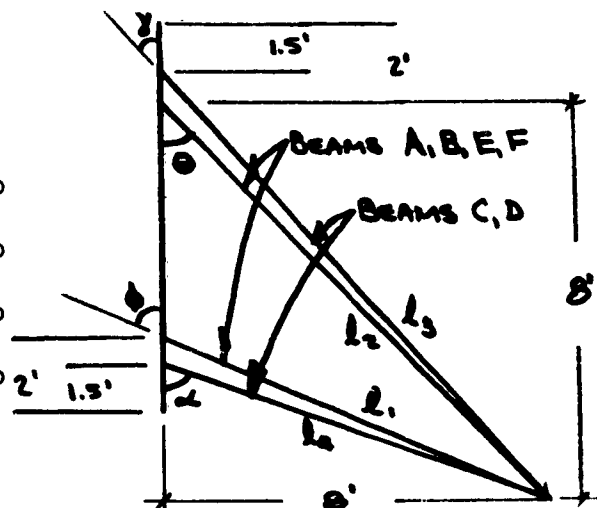


$\Theta = 45°$     $L_2 = 11.3'$
$\phi = 63.4°$     $L_1 = 8.95'$
$\gamma = 43.3°$     $L_3 = 11.7'$
$\alpha = 66.35°$     $L_4 = 8.73$

Design of tie rods

The components of the loads acting on the rods were found above. The resultants of these components will be the loads acting on the tie rods which constitute the design loads.

$$R_{a_1} = R_{f_1} = \sqrt{(R_{ax_1})^2 + (R_{ay_1})^2} = [(7776)^2 + (3890)^2]^{1/2} = 8,690 \text{ lb}$$

$$R_{a_2} = R_{f_2} = \sqrt{(R_{ax_2})^2 + (R_{ay_2})^2} = [(7776)^2 + (7776)^2]^{1/2} = 11,000 \text{ lb}$$

$$R_{b_1} = R_{e_1} = \sqrt{(R_{bx_1})^2 + (R_{by_1})^2} = [(15552) + (7780)^2]^{1/2} = 17,390 \text{ lb}$$

$$R_{b_2} = R_{e_2} = \sqrt{(R_{bx_2})^2 + (R_{by_2})^2} = [(15552)^2 + (15552)^2]^{1/2} = 22,000 \text{ lb}$$

$$R_{c_1} = R_{d_1} = \sqrt{(R_{cx_1})^2 + (R_{cy_1})^2} = [(11664)^2 + (5100)^2]^{1/2} = 12,720 \text{ lb}$$

$$R_{c_2} = R_{d_2} = \sqrt{(R_{cx_2})^2 + (R_{cy_2})^2} = [(11664)^2 + (12400)^2]^{1/2} = 17,100 \text{ lb}$$

Design of Rods $A_1$ and $F_1$

$$A = \frac{P}{F_a} = \frac{8690}{50000} = .1738 \qquad\qquad \text{Bar } \tfrac{1}{2}\,\phi$$

Design of Rods $A_2$ and $F_2$

$$A = \frac{P}{F_a} = \frac{11000}{50000} = .220 \qquad\qquad \text{Bar } \tfrac{9}{16}\,\phi$$

Design of Rods $B_1$ and $E_1$

$$A = \frac{17390}{50000} = .3478 \qquad\qquad \text{Bar } \tfrac{11}{16}\,\phi$$

Design of Rods $B_2$ and $E_2$

$$A = \frac{22000}{50000} = .440 \qquad\qquad \text{Bar } \tfrac{3}{4}\,\phi$$

Design of Rods $C_1$ and $D_1$

$$A = \frac{12720}{50000} = .2544 \qquad\qquad \text{Bar } \tfrac{5}{8}\,\phi$$

Design of Rods $C_2$ and $D_2$

$$A = \frac{17100}{50000} = .342 \qquad\qquad \text{Bar } \tfrac{11}{16}\,\phi$$

Calculation of moments on frame by moment distribution method

| | 1 | 2 | 2' | 3 | | | 2' | 4 | | | 4' | 5 | | | 2' | 6 | | | 2' | 7 | 8' | 8 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DF | | .53 | .47 | .50 | .50 | | | .50 | .50 | | | .50 | .50 | | | .50 | .50 | | | .47 | .53 | |
| | | -14.415 | | +3.387 | | | | +4.547 | | | | -4.547 | | | | | | | | +14.162 | | |
| FEM | +10.368 | +1.137 | -1.137 | +1.137 | +1.137 | | -1.137 | +4.547 | +1.137 | +1.137 | | -4.547 | +1.137 | +1.137 | | -1.137 | +1.137 | +1.137 | | +14.162 | +10.368 | 0 |
| | -10.368 | -10.368 | | | | | | | | | | | | | | | | | | .47 .53 | +5.184 | +10.368 |
| | | -5.184 | | | | | | | | | | | | | | | | | | -1.137 | | |
| | | +7.640 | +6.775 | +3.387 | | | | | | | | | | | | | | | | | | |
| | | -.846 | -.846 | -1.693 | -1.693 | | -.846 | -1.282 | | -.641 | -.641 | | | | | | | | | -.253 | | |
| | | | | +3.387 | -1.693 | | | | +1.012 | | | -1.282 | +2.025 | +1.012 | | -.506 | -.253 | | | -6.656 | -7.506 | |
| | | | | -1.693 | -.641 | | -1.282 | +1.012 | -.506 | | +2.025 | -.641 | | | | -3.328 | | | | +.832 | | |
| | | +.214 | +.297 | +.429 | +.148 | | | +.214 | | -.217 | | +.832 | +.832 | +1.664 | | +1.664 | | | | | | |
| | | -.037 | -.037 | -.074 | -.074 | | | +.429 | -.217 | +.429 | +.434 | +2.025 | -.289 | -.144 | | -.144 | | | | | | |
| | +.335 | | | | | | | -.037 | -.074 | -.044 | +.214 | -.289 | -.217 | | | | | | | | | |
| | | +.014 | +.214 | | | | | -.088 | | -.044 | | | | | | | | | | | | |
| | | +.011 | +.028 | +.028 | +.005 | | | -.088 | -.044 | | -.037 | -.044 | +.130 | +.065 | | +.065 | +.039 | +.039 | | +.019 | +.451 | |
| | | -.001 | -.001 | -.005 | -.002 | | | +.065 | +.130 | | -.088 | +.130 | +.019 | +.039 | | -.200 | -.400 | +.100 | | +.400 | | |
| | +.012 | +.012 | | +.012 | -.002 | | +.014 | | | -.024 | +.014 | -.034 | +.050 | +.100 | | +.100 | +.050 | | | +.050 | | |
| | | | | +.028 | -.003 | | | -.024 | +.130 | | | -.012 | +.034 | -.017 | | +.017 | | | | | | |
| | | | | -.002 | | | -.006 | +.014 | -.024 | | +.001 | -.034 | | | | | | | | | | |
| | | | | -.003 | +.001 | | +.003 | -.006 | +.003 | | -.006 | -.012 | -.003 | +.007 | | +.003 | +.007 | | | +.003 | -.028 | |
| | | | | | | | | +.003 | +.007 | | +.003 | +.007 | +.007 | +.003 | | +.007 | +.003 | +.006 | | +.025 | | |
| | | | | | | | -.001 | -.001 | +.003 | | +.003 | -.003 | | +.003 | | -.012 | +.006 | | | +.003 | | |
| | | | | | | | -.001 | +.003 | | | -.003 | -.003 | | | | +.001 | -.001 | | | | | |
| | | | | +.001 | +.001 | | -.001 | -.001 | | | -.001 | -.003 | | | | | | | | -.001 | -.002 | |
| Final | -7.565 | +7.564 | -1.091 | +1.092 | +.001 | | -3.628 | +3.630 | -3.628 | -3.627 | | +3.627 | | +1.091 | | -1.093 | | | | -7.565 | +7.565 | |
| | 0 | | | | +.001 | | | | | | | | | | | | | | | -7.565 | +7.565 | 0 |

Assume $\dfrac{I_{12}}{I_{24}} = 6$  $\qquad \dfrac{I_{45}}{I_{24}} = 2$  $\qquad\qquad\qquad \dfrac{.563}{1.063} = .53$

$K_{12} = \dfrac{6}{8}(\dfrac{3}{4}) = .563$  $\qquad K_{23} = \dfrac{1}{2} = .50$  $\qquad K_{45} = \dfrac{2}{4} = .50$  $\qquad \dfrac{.50}{1.063} = .47$

Calculation of unknowns on Beam G by superposition
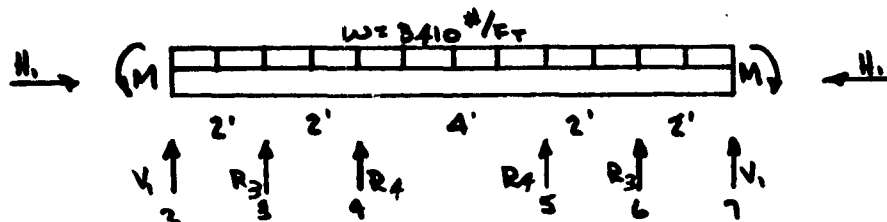


Section 2 - 3
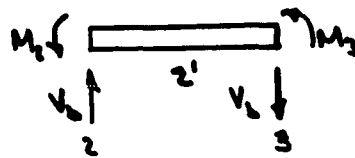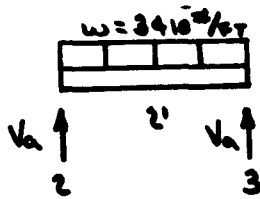


$V_a = \dfrac{1}{2}(3410)\,2$  $\qquad (M_2 + M_3)\dfrac{1}{2} = V_b$  $\qquad V_1 = V_a + V_b$

$V_a = 3410 \text{ lb}$  $\qquad V_b = \dfrac{1}{2}(+7,565 + 1,092)$  $\qquad V_1 = 3410 + 4328$

$\qquad\qquad\qquad\quad V_b = 4,328 \text{ lb}$  $\qquad V_1 = 7,738 \text{ lb}$

Section 3 - 4
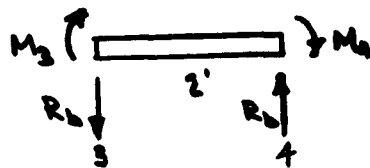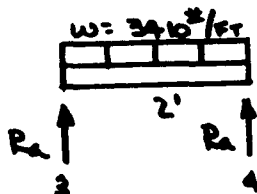


$R_a = \dfrac{1}{2}(3410)2$  $\qquad R_b = \dfrac{1}{2}(M_3 + M_4)$  $\qquad R_3 = V_a + R_a - V_b - R_b$

$R_a = 3410 \text{ lb}$  $\qquad R_b = \dfrac{1}{2}(1,092 + 3,628)$  $\qquad R_3 = 3410 + 3410 -$
$\qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad 4328 - 2360$

$\qquad\qquad\qquad\qquad R_b = 2360 \text{ lb}$  $\qquad R_3 = 132 \text{ lb}$

Section 4 - 5
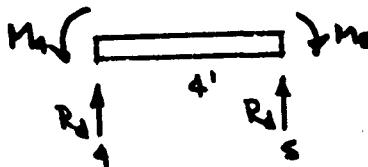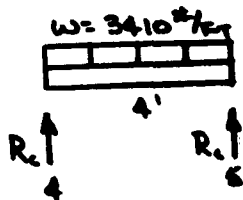


$R_c = \frac{1}{2}(3410)(4)$

$R_c = 6820 \text{ lb}$

$R_d = \frac{1}{4}(M_4 + M_5)$

$R_d = \frac{1}{4}(3,628 - 3,628)$

$R_d = 0$

$R_4 = R_a + R_b + R_c + R_d$

$R_4 = 3410 + 2360 + 6820$

$R_4 = 12,590 \text{ lb}$

Calculation of unknowns on Beam A in Y-Z plane

$\Sigma F_y = 0 = V_1 + R_{ay_2} + R_{ay_1} - R_{ay}$

$R_{ay} = V_1 + R_{ay_2} + R_{ay_1}$

$R_{ay} = 7,738 + 7,776 + 3,890$

$\underline{R_{ay} = 19,404 \text{ lb}}$

$\overset{\curvearrowright}{\Sigma M_0} = -M - 8R_{az} + \frac{1}{2}(1944)(8)^2 = 0$

$R_{az} = \frac{1}{8}[62,208 - 7,564]$

$\underline{R_{az} = 6,830 \text{ lb}}$

$\Sigma F = 0 = H_1 + R_{az} - 1944(8)$

$H_1 = 15,552 - 6830$

$\underline{H_1 = 8722 \text{ lb}}$

$\underline{M_{max.} = 11,973 \text{ ft-lb}}$
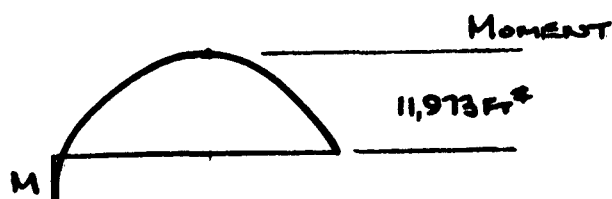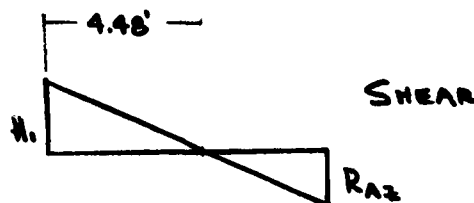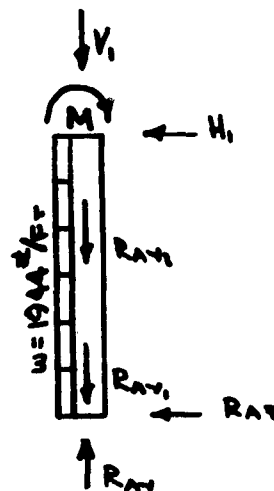
Calculation of unknowns on Beam A in X-Y plane

To design beam, the max.
bending moment must be
calculated. This is done
by the shear and moment
diagrams at right.

$$M_{max.} = \frac{wL^2}{32} = \frac{1944(8)^2}{32}$$

$\underline{M_{max.} = 3888 \text{ ft-lb}}$

Calculation of unknowns on Beam B in X-Y plane

$\Sigma F_y = 0 = R_3 + R_{by_2} + R_{by_1} - R_{by}$

$R_{by} = 132 + 15,552 + 7780$

$\underline{R_{by} = 23,464 \text{ lb}}$
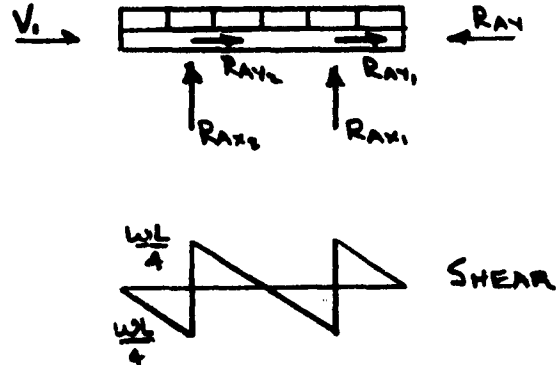
The maximum bending moment for this beam will occur at the same
point as Beam A in X-Y plane since both beams are loaded in the
same manner.

$$M_{max.} = \frac{wL^2}{32} = \frac{3888(8)^2}{32}$$

$\underline{M_{max.} = 7776 \text{ ft-lb}}$

Calculation of unknowns on
Beam C in X-Y plane

$\Sigma F_y = R_4 + R_{cy_2} + R_{cy_1} - R_{cy} = 0$

$R_{cy} = 12590 + 12400 + 5100$

$\underline{R_{cy} = 30,090 \text{ lb}}$

$\underline{M_{max.} = 6076 \text{ ft-lb}}$
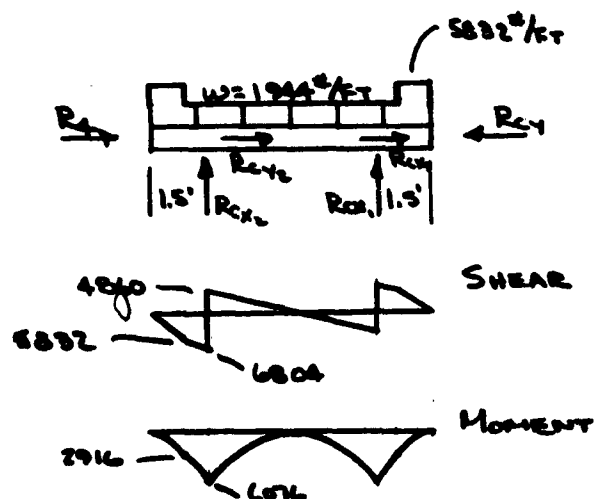
Design of Beam G

Maximum bending moment $\quad M = 7,564$ ft-lb

Axial compressive load $\quad H_1 = 8,722$ lb

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

$$\frac{1}{50,000}\left[\frac{8722}{A} + \frac{7564(12)}{S_{xx}}\right] \leq 1$$

$$\frac{.1744}{A} + \frac{1.815}{S_{xx}} \leq 1$$

Try L 6 x 4 x 5/16 $\quad S_{xx} = 2.8$ in.$^3 \quad I_{xx} = 11.4$ in.$^4 \quad A = 3.03$ in.$^2$

$$\frac{.1744}{3.03} + \frac{1.815}{2.8} = .058 + .648 = .706 < 1$$

Use L 6 x 4 x 5/16

To strengthen the 4' section on Beam G as assumed in the moment distribution method, use another L 6 x 4 x 5/16 butting the 4" angles together. This will give the required I.

Design of Beam A

Maximum bending moment in X-Y plane $\quad M_{max.} = 3,888$ ft-lb

Maximum bending moment in Y-Z plane $\quad M_{max.} = 11,973$ ft-lb

Axial compressive load $\quad R_{ay} = 19,404$ lb

$$\frac{f_a}{F_a} + \left(\frac{f_b}{F_b}\right)_{xx} + \left(\frac{f_b}{F_b}\right)_{yy} \leq 1$$

$$\frac{1}{50,000}\left[\frac{19404}{A} + \frac{11973(12)}{S_{xx}} + \frac{3888(12)}{S_{yy}}\right] \leq 1$$

$$\frac{.388}{A} + \frac{2.87}{S_{xx}} + \frac{.933}{S_{yy}} \leq 1$$

Try 10 B 15 $\quad A = 4.40$ in.$^2 \quad S_{xx} = 13.8$ in.$^3 \quad I_{xx} = 68.8$ in.$^4$

$\quad S_{yy} = 1.39$ in.$^3$

$$\frac{.388}{4.40} + \frac{2.87}{13.8} + \frac{.933}{1.39} = .088 + .208 + .671 = .967 < 1$$

Ratio of I's for $\dfrac{10\ B\ 15}{L\ 6\ x\ 4\ x\ 5/16} = \dfrac{68.8}{11.4} = 6.04 \approx 6$ as assumed

Use 10 B 15
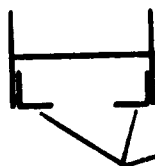
Use L 1-1/2 x 1-1/2 x 3/16 to sustain bearing loads

Design of Beam B

Maximum bending moment   $M_{max.} = 7,776$ ft.-lb
Axial compressive load   $R_{by} = 23,464$ lb

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

$$\frac{1}{50,000}\left[\frac{23,464}{A} + \frac{7776(12)}{S_{xx}}\right] \leq 1$$

$$\frac{.469}{A} + \frac{1.866}{S_{xx}} \leq 1$$

Try 4 ⌴ 7.25   $A = 2.12$ in.$^2$   $S_{xx} = 2.3$ in.$^3$

$$\frac{.469}{2.12} + \frac{1.866}{2.3} \quad .221 + .811 = 1.032 > 1$$

Try 4 I 7.7   $A = 2.21$ in.$^2$   $S_{xx} = 3.0$ in.$^3$

$$\frac{.469}{2.21} + \frac{1.866}{3.0} = .212 + .622 = .834 < 1$$

Use 4 I 7.7

Design of Beam C

Maximum bending moment   $M_{max.} = 6076$ ft-lb
Axial compressive load   $R_{cy} = 30,090$ lb

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

$$\frac{1}{50,000} \left[ \frac{30090}{A} + \frac{6076(12)}{S_{xx}} \right] \leq 1$$

$$\frac{.602}{A} + \frac{1.458}{S_{xx}} \leq 1$$

Try 4 I 7.7 $\qquad$ A = 2.21 in.$^2$ $\qquad$ $S_{xx}$ = 3.0 in.$^3$

$$\frac{.602}{2.21} + \frac{1.458}{3.0} = .272 + .486 = .758 < 1$$

Use 4 I 7.7

Design of Struts $\qquad$ Axial Load $R_{az}$ = 6830 lb

$$A = \frac{P}{S} = \frac{6830}{50,000} = .1366 \text{ in.}^2$$

Use L 1 x 1 x 1/8 $\qquad$ A = .23 in.$^2$ $\qquad$ .80 lb/ft

Design of timber 1' wide to withstand end loading. To be placed in the 4' 2" x 8' opening above and below shelter entranceway.

$$M = \frac{wL^2}{8}$$

$$M = \frac{(1944)(4.5)^2(12)}{8}$$

M = 59,000 in.-lb

$$f = \frac{M}{S} = 6000 \text{ psi}$$

$$S = \frac{59,000}{6,000} = 9.84 \text{ in.}^3$$

$$S = \frac{bh^2}{6} \qquad b = 12"$$

$$h = \sqrt{\frac{6S}{b}} = \sqrt{\frac{6(9.84)}{12}} = 2.22 \text{ in.}$$

Try 3" x 12" $\qquad$ h = 2.625 in.
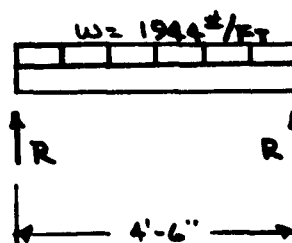
Shear $\qquad H = \dfrac{3R}{2bh}$

$\qquad R = 1/2(1944)(4.5) = 4374 \text{ lb}$

$\qquad H = \dfrac{3(4374)}{2(12)(2.625)} = 208 \text{ psi} < 640$

Use 3" x 12"

## Entrance End



w = 13,900 lb/ft
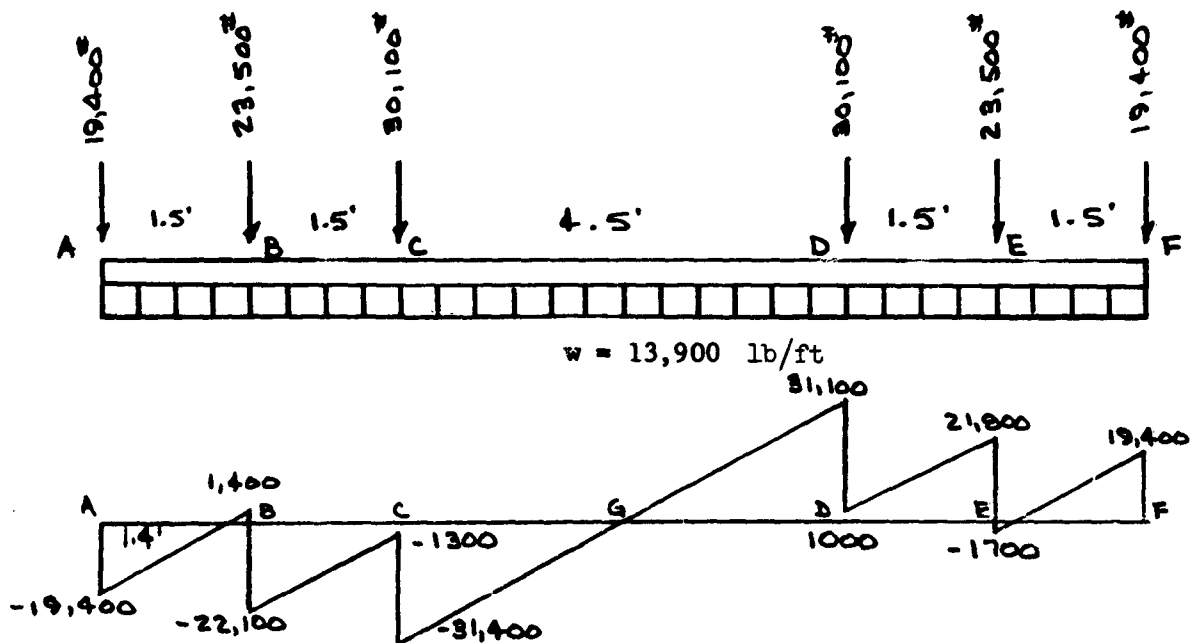


Max. M = $M_G = \frac{1}{2}$ x 19,400 x 1.4 + $\frac{1}{2}$ x 22,100 x 1.5 + 31,400 x $\frac{1}{2}$ x 2.25 =

$\qquad$ 13,600 + 16,600 + 35,400

$M_G$ = 65,600 ft-lb = 788,000 in.-lb

Splice sills with 4.5' long channels

$M_c$ = 13,600 + 16,600 = 30,200 ft-lb = 362,000 in.-lb

$S_T = \dfrac{362,000}{6000} = 60.4$

$S_{\mathsf{E}} = \dfrac{M}{f} = \dfrac{788,000 - 362,000}{50,000} = \dfrac{326,000}{50,000} = 6.52$

Try b = 18"

$S = 60.4 = \dfrac{bh^2}{6}$   $\qquad h^2 = \dfrac{60.4 \times 6}{17.5} = 20.7$

h = 4.56"   $\qquad$ Try 18" x 5"

$H = \dfrac{3V}{2bh} = \dfrac{3 \times 31,400}{2 \times 17.5 \times 4.5} = 598 < 640$

2 - 6 ⊔ 8.2's   $\qquad$ S = 8.6

   Use 18" x 5" x 12'-0", reinf. center section with two
   6 ⊔ 8.2 lb/ft 4.5' long

Bearing plates

10 B 15   $A_{min.} = \dfrac{19,404}{1100} = 17.6$ sq in.

4 I 7.7   $A_{min.} = \dfrac{30,090}{1100} = 27.4$ sq in.

$\qquad A_{min.} = \dfrac{23,464}{1100} = 21.4$ sq in.

$\qquad$ 10 B 15   use 11" x 5" x 1/4"   PL

$\qquad$ 4 I 7.7   use  6" x 5" x 1/4"   PL
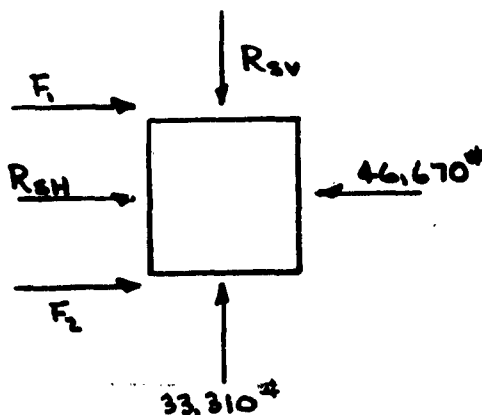
Try 15" x 15"    x - section
$\phi_3 = 30°$

N  = Normal force = w h A = 100 x 14 x 1.25 x 2.855

N  = 5000 lb

$F_1 = F_2 = N \tan \phi = 5000 \tan 30° = 2880$ lb

Let $P_{ALL.}$ = 3000 psf

$R_{sh}$ = 3000 x 2.885 x 1.25 = 10,800 lb

$F_{rh}$ = 10,800 + 2 x 2880 = 10,800 + 5760 = 16,560

2N = 2 w h A = 2 x 100 x 14 x 2.885 x b = 8090 lb

$R_{sh}$ = 3000 x 2.885 x d = 8650 d

$F = 2 N \tan \phi = 8090 \tan 30°$ x b = 4660 b

$F + R_{sh}$ = 8650 d + 4660 b = 46,670      Let b = d

     13,310 d = 46,670     d = b = 3.51'

$R_{sv}$ = 3000 x 3.51 x 2.885 + 4045 x 3.51 =

$R_{sv}$ = 30,400 lb + 14,200 = 44,600 lb > 33,100

<u>Use 18" x 18" x 13'-0" reinf. conc.</u>

Based on Army TM 5-311, page 99, actual field tests show this to be adequate.  Use same design for entrance end.

#3 TIE RODS

#4 BARS

18"

18"

3"

Deadman (2)



2'

4'

45°

9.9'

3'

2'

46.8°

7.63'

h= 14'

7'

$$F_x = 25{,}350 \cos 23.2° + 32{,}980 \cos 45°$$
$$F_x = 23{,}320 + 23{,}350 = 46{,}670 \text{ lb}$$
$$F_y = 25{,}350 \sin 23.2° + 32{,}980 \sin 45°$$
$$F_y = 9960 + 23{,}350 = 33{,}310 \text{ lb}$$
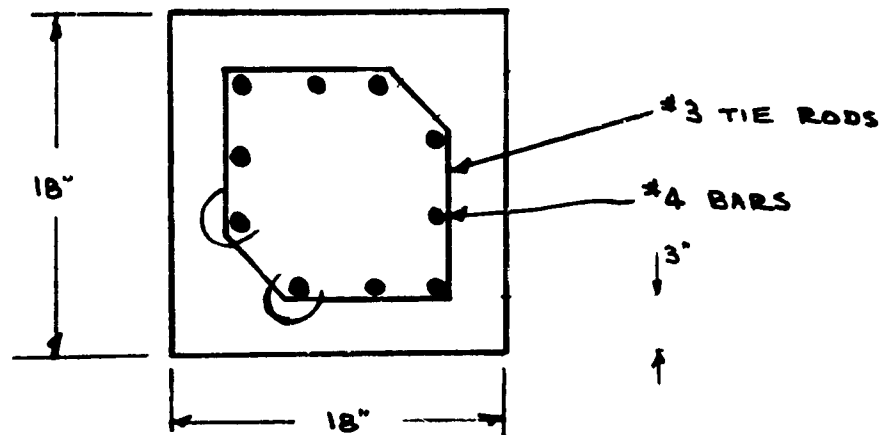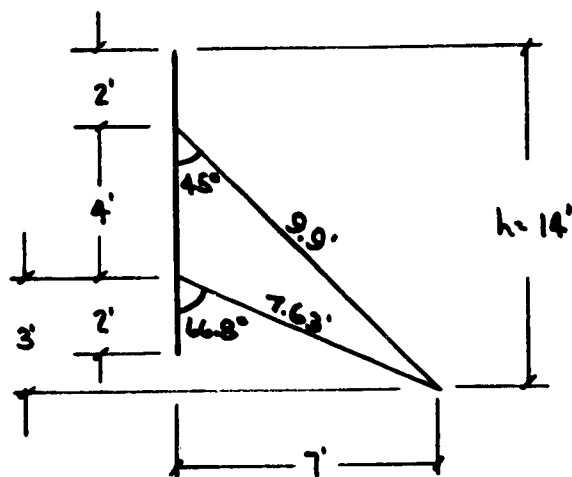
## ENTRANCEWAY

Effect of Earth Friction on Entranceway

Referring to the plan view of the entranceway an unbalanced force will occur due to the side loading on the frame I-G. There is no force acting at the opening of the shelter to counterbalance the loading on I-G. Since only a minimum of movement can be tolerated, the loading on I-G must be balanced naturally or by attaching a deadman to frame I-G.

First the natural forces which would act on the entrance with impending motion will be investigated.

Since we are assuming impending motion

$$F = N \tan \phi$$

Where N = normal force
F = friction force

For horizontal surfaces

$\tan \phi$ = coefficient of static friction
$\phi = 30°$ for cohesionless soil

In our case N = whA

Where w = 100 lb/ft³
h = Depth of soil
A = Area at h
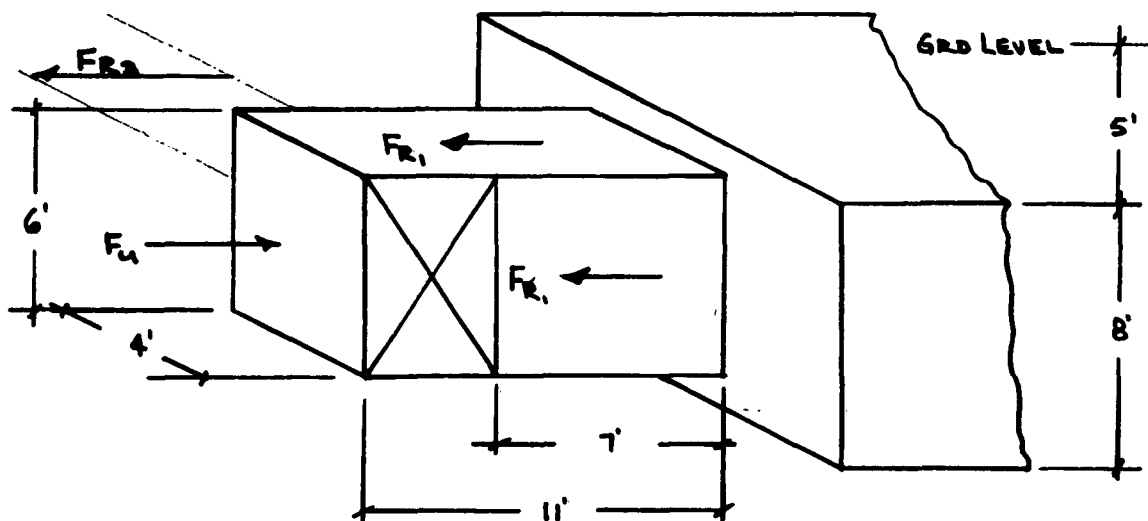
$F_{r_2} = N \tan \phi \qquad N = (100 \text{ lb/ft}^3)(6 \text{ ft})(11 \text{ ft})(4 \text{ ft}) = 26,400 \text{ lb}$

$F_{r_2} = (26,400 \text{ lb})(\tan 30°)$

$F_{r_2} = 15,230 \text{ lb}$

$F = w \, h \, A \tan^2 (45° + \dfrac{\phi}{2}) \qquad$ for vertical surfaces

$\qquad A = dh \, L \qquad L = \text{Length of surface}$

$F = \displaystyle\int_{6}^{12} w \, h \, L dh \, \tan^2(30°)$

$\quad = \dfrac{1}{2} w \, L \tan^2 (30°)[\, h^2 \,]_{6}^{12} = \dfrac{1}{2} w \, L \tan^2(30°) \, [144-36]$

$\quad = \dfrac{1}{2} w \, L \, (.334)(108)$

$F_{r_1} = \dfrac{1}{2}(100)(7)(.334)(108)$

$F_{r_1} = 12,620$

$F_{r_2} = \dfrac{11}{7}(12,620)$

$F_{r_2} = 19,820$

Total friction force = $F_r$ = 15,230 + 19,820 + 12,620 = 61,670 lb

$F_u$ = Unbalanced force = (1945 psf)(4 ft)(6 ft) = 46,680 lb

Since $F_u < F_r$ no movement will occur in the entrance due to the
unbalanced force $F_u$.

Loading on Frames A-A', J-J', L-L'
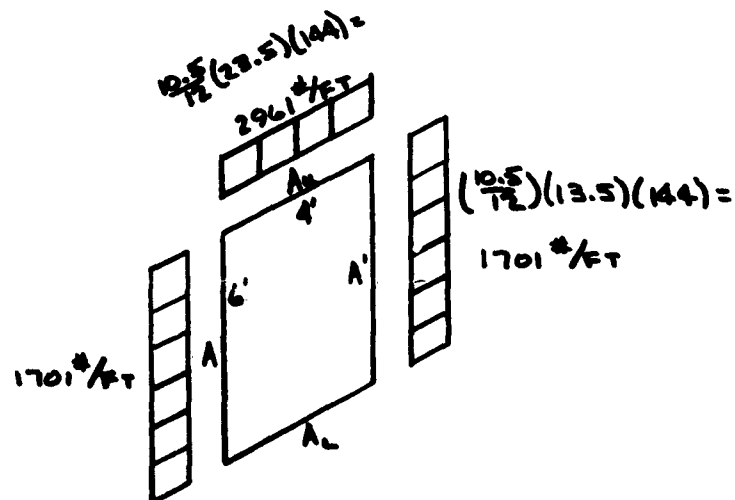
$A_u$ is rigidly connected to A and A'

$A_L$ is simply connected to A and A'

Assume $\quad \dfrac{I_a}{I_{a_u}} = 2.4$

$K_a = \dfrac{2.4}{6} \quad \dfrac{3}{4} = .3 \qquad\qquad \dfrac{.3}{.55} = .55$

$K_{a_u} = \dfrac{1}{4} = .25 \qquad\qquad\quad \dfrac{.25}{.55} = .45$

Moment Distribution

|  |  | −3.706 |  |  | +4.540 |  |  |
|---|---|---|---|---|---|---|---|
| 0 | .55 | .45 |  |  | .45 | .55 | 0 |
| +5.103 | −5.103 | +3.948 |  | −3.948 | +5.102 | −5.103 |  |
| −5.103 | −2.551 |  |  |  | +2.551 | +5.103 |  |
|  | +2.038 | +1.668 |  | + .834 |  |  |  |
|  |  | −1.021 |  | −2.043 | −2.497 |  |  |
|  | + .562 | + .459 |  | + .229 |  |  |  |
|  |  | − .051 |  | − .103 | − .126 |  |  |
|  | + .028 | + .023 |  | + .011 |  |  |  |
|  |  | − .002 |  | − .005 | − .006 |  |  |
|  | + .001 | + .001 |  |  |  |  |  |
| 0 | −5.025 | +5.025 |  | −5.025 | +5.025 |  | 0 |

Calculation of unknowns on Beam A and A'

$\Sigma F = 0 = R_{a_y} - V_1$

$\qquad R_{a_y} = V_1 = \dfrac{1}{2}(2961)(4)$

$\qquad \underline{R_{a_y} = 5922 \text{ lb}}$

$\overset{\curvearrowright}{\Sigma} M_0 = -M - 6H + \dfrac{1}{2}(6)^2(1701) = 0$

$\qquad H = \dfrac{1}{6}[30,618 - 5,025]$

$\qquad \underline{H = 4266 \text{ lb}}$

$\Sigma F = 0 = (1701)(6) - H - H_1$

$\qquad H_1 = 10,206 - 4266$

$\qquad \underline{H_1 = 5940 \text{ lb}}$

$\underline{M = M_{max.} = 5,025 \text{ ft-lb}}$



SHEAR

MOMENT

274
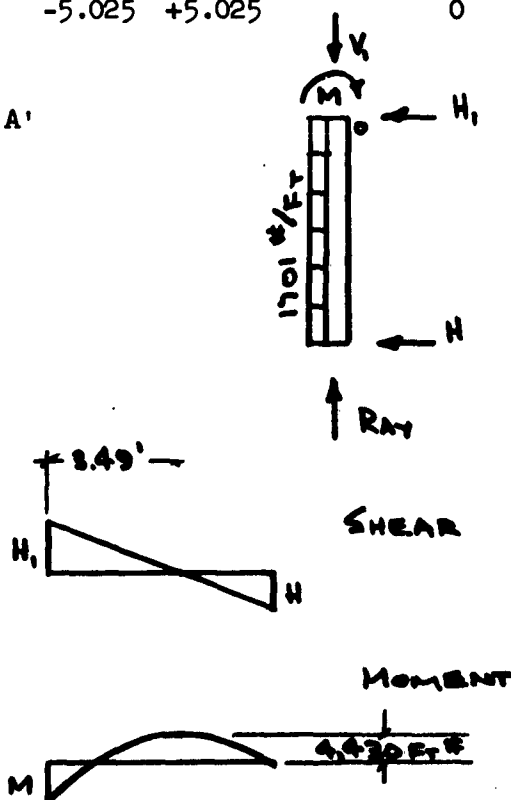
Design of Beam $A_u$, $I_u$, $L_u$

Maximum bending moment $\qquad$ M = 5,025 ft-lb

Axial compressive load $\qquad$ $H_1$ = 5940 lb

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

$$\frac{1}{50,000}\left[\frac{5940}{A} + \frac{5025\ (12)}{S_{xx}}\right] \leq 1$$

$$\frac{.1188}{A} + \frac{1.206}{S_{xx}} \leq 1$$

Try 3 ⌴ 6 $\qquad$ A = 1.75 in.$^2$ $\qquad$ $S_{xx}$ = 1.4 in.$^3$ $\qquad$ $I_{xx}$ = 2.1 in.$^4$

$$\frac{.1188}{1.75} + \frac{1.206}{1.4} = .068 + .861 = .929 < 1$$

Design of Beam A and A', J and J', L and L'

Maximum bending moment $\qquad$ $M_{max.}$ = 5025 ft-lb

Axial compressive load $\qquad$ $V_1$ = 5922 lb

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

$$\frac{1}{50,000}\left[\frac{5922}{A} + \frac{5025(12)}{S_{xx}}\right] \leq 1$$

$$\left[\frac{.1184}{A} + \frac{1.206}{S_{xx}}\right] \leq 1$$

Try 3 ⌴ 6 $\qquad$ A = 1.75 in.$^2$ $\qquad$ $S_{xx}$ = 1.4 in.$^3$ $\qquad$ $I_{xx}$ = 2.1 in.$^4$

$$\frac{.1184}{1.75} + \frac{1.206}{1.4} = .068 + .861 = .929 < 1$$

$$\frac{I_a}{I_{a_u}} = \frac{2.1}{2.1} = 1 < 2.4 \quad \text{the original assumption}$$

Moment distribution must be recalculated. This time use $\dfrac{I_a}{I_{a_u}} = 1$

Moment Distribution

| 0 | .33 | .67 | | .67 | .33 | 0 |
|---|---|---|---|---|---|---|
| | -3.706 | | | | | |
| +5.103 | -5.103 | +3.948 | -3.948 | +5.103 | | -5.103 |
| -5.103 | -2.551 | | | | +2.551 | +5.103 |
| | +1.235 | +2.471 | +1.235 | | | |
| | | -1.647 | -3.294 | -1.647 | | |
| | + .549 | +1.098 | + .549 | | | |
| | | - .183 | - .366 | - .183 | | |
| | + .061 | + .122 | + .061 | | | |
| | | - .020 | - .041 | - .020 | | |
| | + .007 | + .013 | + .006 | | | |
| | | - .002 | - .004 | - .002 | | |
| | | + .001 | | | | |
| 0 | -5.802 | +5.801 | -5.802 | +5.802 | | 0 |

$$K_a = \frac{1}{6}\ \frac{3}{4} = \frac{1}{8} = .125 \qquad \frac{.125}{.345} = .33$$

$$K_{a_u} = \frac{1}{4} = .25 \qquad \frac{.25}{.375} = .67$$

Recalculation of unknowns on Beams A and A'

$$\Sigma F = 0 = R_{a_y} - V_1$$

$$R_{a_y} = \frac{1}{2}(2961)(4)$$

$$\underline{R_{a_y} = 5922\ lb}$$

$$\circlearrowleft \Sigma M_0 = 0 = -M - 6H + \frac{1}{2}(6)^2(1701)$$

$$H = \frac{1}{6}(30618 - 5802)$$

$$\underline{H = 4136\ lb}$$

$$\Sigma F = 0 = 1701(6) - H - H_1$$

$$H_1 = +0206 - 4136$$

$$\underline{H_1 = 6070\ lb}$$

$$\underline{M_{max.} = M = 5802\ ft\text{-}lb}$$



SHEAR

MOMENT

5034 FT⁻#
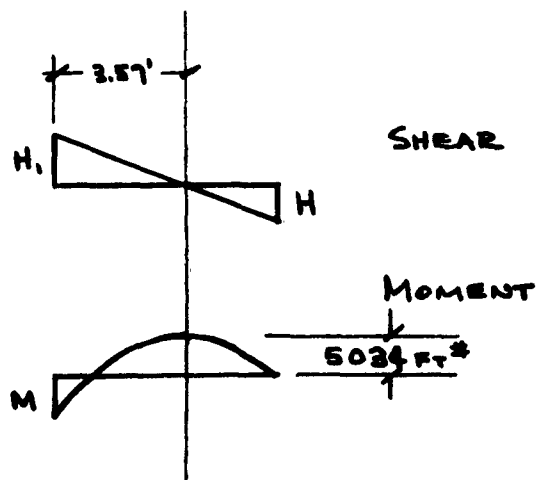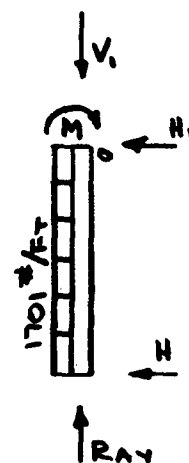
Redesign of Beam $A_u$, $J_u$, $L_u$

Maximum bending moment $\qquad M = 5,802$ ft-lb

Axial compressive load $\qquad H_1 = 6070$ lb

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

$$\frac{1}{50,000}\left[\frac{6070}{A} + \frac{5802(12)}{S_{xx}}\right] \leq 1$$

$$\frac{.1214}{A} + \frac{1.392}{S_{xx}} \leq 1$$

Try 3 I 5.7 $\qquad A = 1.64$ in.$^2$ $\qquad S_{xx} = 1.7$ in.$^3$ $\qquad I_{xx} = 2.5$ in.$^4$

$$\frac{.1214}{1.64} + \frac{1.392}{1.7} = .074 + .820 = .894 < 1$$

$$\frac{I_a}{I_{a_u}} = \frac{2.5}{2.5} = 1 \quad \text{as assumed 2nd time}$$

Use 3 I 5.7


Redesign of Beams A and A', J and J', L and L'

Maximum bending moment $\qquad M_{max.} = 5,802$ ft-lb

Axial compressive load $\qquad R_{ay} = 5922$ lb

$$\frac{1}{50,000}\left[\frac{5922}{A} + \frac{5802(12)}{S_{xx}}\right] \leq 1$$

Try 3 I 5.7 $\qquad A = 1.64$ in.$^2$ $\qquad S_{xx} = 1.7$ in.$^3$ $\qquad I_{xx} = 2.5$ in.$^4$

$$\frac{.1184}{1.64} + \frac{1.392}{1.7} = .072 + .820 = .892 < 1$$

Use 3 I 5.7

Design of Strut $A_L$

$$A = \frac{P}{S} = \frac{4136}{50,000}$$

$$A = .0827 \text{ in.}^2$$

Use L 1 x 1 x 1/8 $\qquad A = .23$ in.$^2$ $\qquad$ .80 lb/ft

Loading on Frames B-B', C-C', D-D', K-K'
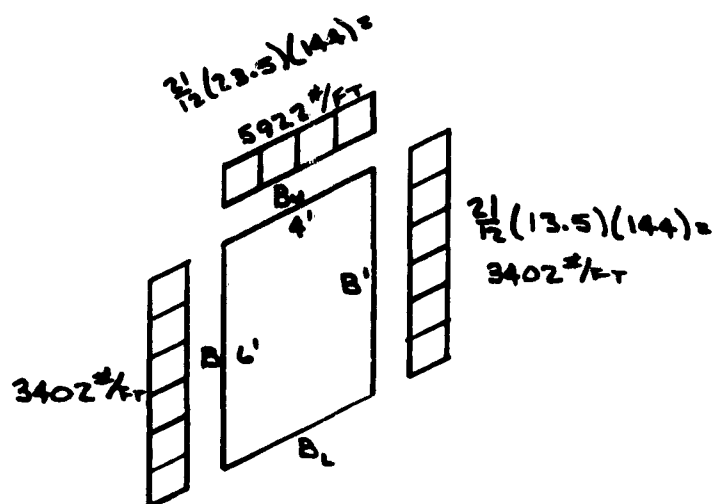


$A_u$ is rigidly connected to B and B'

$A_L$ is simply connected to B and B'

Assume $\dfrac{I_A}{I_{A_u}} = 1$

$K_B = \dfrac{1}{6}\,\dfrac{3}{4} = .125 \qquad \dfrac{.125}{.375} = .33$

$K_{B_u} = \dfrac{1}{4} = .25 \qquad \dfrac{.25}{.375} = .67$

Moment Distribution

| 0 | | .33 | .67 | | .67 | .33 | | 0 |
|---|---|---|---|---|---|---|---|---|
| +10.206 | | -10.206 | + 7.896 | | - 7.896 | +10.206 | | -10.206 |
| -10.206 | | - 5.103 | | | | + 5.103 | | +10.206 |
| | | + 2.471 | + 4.942 | | + 2.471 | | | |
| | | | - 3.294 | | - 6.589 | - 3.295 | | |
| | | + 1.098 | + 2.196 | | + 1.098 | | | |
| | | | - .366 | | - .732 | - .366 | | |
| | | + .122 | + .244 | | + .122 | | | |
| | | | - .041 | | - .082 | - .041 | | |
| | | + .014 | + .028 | | + .014 | | | |
| | | | - .005 | | - .009 | - .005 | | |
| | | + .002 | + .003 | | + .001 | | | |
| 0 | | -11.602 | +11.603 | | -11.602 | +11.602 | | 0 |

Calculation of unknowns on Beam B and B'

$$\Sigma F_y = V_1 - R_{b_y} = 0$$

$$R_{b_y} = \tfrac{1}{2}(5922)(4)$$

$$\underline{R_{b_y} = 11844 \text{ lb}}$$

$$\Sigma^+ M_0 = 0 = -6H - M + \tfrac{1}{2}(6)^2(3402)$$

$$H = \tfrac{1}{6}[61236 - 11,602]$$

$$\underline{H = 8272 \text{ lb}}$$

$$\Sigma F = 0 = H_1 + H - (6)(3402)$$

$$H_1 = 20412 - 8272$$

$$\underline{H_1 = 12,140 \text{ lb}}$$

$$\underline{M_{max.} = M = 11,602 \text{ ft-lb}}$$
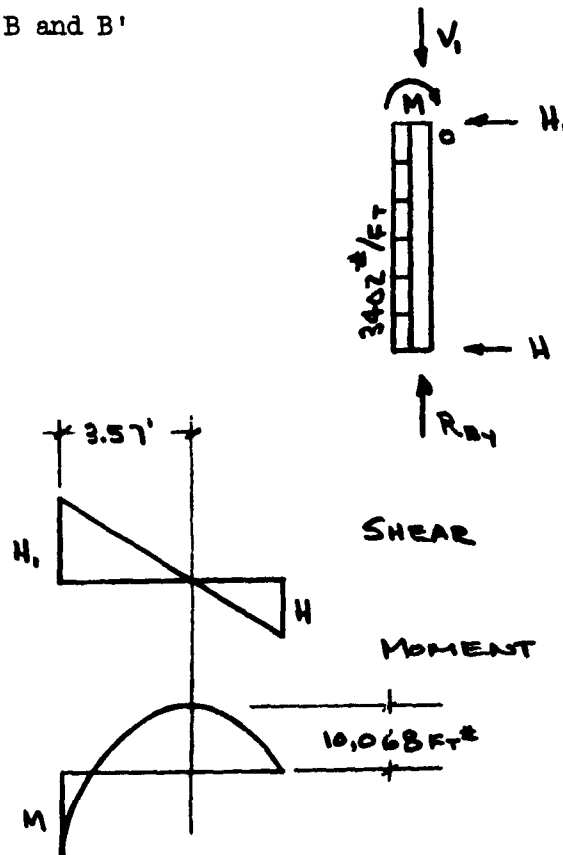
Design of Beams B and B', C and C', D and D', K and K'

Maximum bending moment $\quad M_{max.} = 11,602$ ft-lb
Axial compressive load $\quad R_{b_y} = 11,844$ lb

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

$$\frac{1}{50,000}\left[\frac{11,844}{A} + \frac{11,602(12)}{S_{xx}}\right] \leq 1$$

$$\frac{.2368}{A} + \frac{2.78}{S_{xx}} \leq 1$$

Try 4 I 9.5 $\quad$ A = 2.76 in.$^2$ $\quad$ $S_{xx}$ = 3.3 in.$^3$ $\quad$ $I_{xx}$ = 6.7 in.$^4$

$$\frac{.2368}{2.76} + \frac{2.78}{3.3} = .086 + .842 = .928 < 1$$

$$\frac{I_a}{I_{a_u}} = \frac{6.7}{6.7} = 1 \quad \text{as assumed}$$

Use 4 I 9.5

Design of Beams $B_u$, $C_u$, $D_u$, $K_u$

Maximum bending moment $\qquad$ M = 11,602 ft-lb
Axial compressive load $\qquad$ $H_1$ = 12,140 lb

$$\frac{1}{50,000}\left[\frac{12140}{A} + \frac{11,602(12)}{S_{xx}}\right] \leq 1$$

$$\frac{.2428}{A} + \frac{2.78}{S_{xx}} \leq 1$$

Try 4 I 9.5 $\qquad$ A = 2.76 in.$^2$ $\qquad$ $S_{xx}$ = 3.3 in.$^3$ $\qquad$ $I_{xx}$ = 6.7 in.$^4$

$$\frac{.2428}{2.76} + \frac{2.78}{3.3} = .088 + .842 = .930 < 1$$

Use 4 I 9.5

Design of Beams $B_L$, $C_L$, $D_L$, $K_L$

Axial compressive load $\qquad$ H = 8,272 lb

$$A = \frac{H}{S} = \frac{8272}{50,000}$$

$$A = .1654 \text{ in.}^2$$

Use L 1 x 1 x 1/8 $\qquad$ A = .23 in.$^2$ $\qquad$ .80 lb/ft

$F_1 = (1944 \text{ psf})(1')(6')(\frac{1}{2})$          $R' = \frac{1}{4}(4')(6')(1944 \text{ psf})$
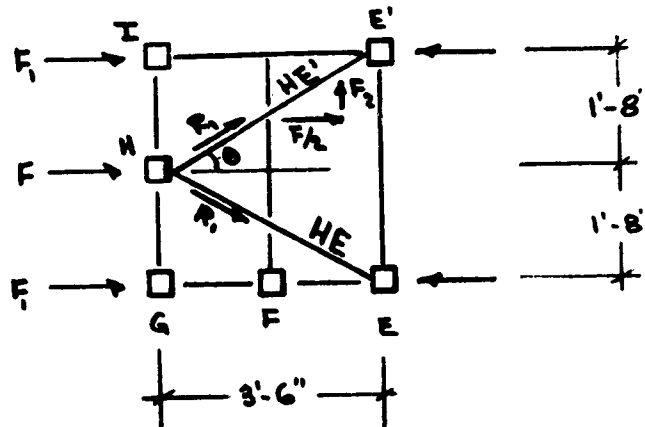
$F_1 = 5832 \text{ lb}$          $R' = 11,664 \text{ lb}$

$F = 2F_1 = 11,664 \text{ lb}$

$\frac{1}{2}F = R_1 \cos \theta$

$\theta = \tan^{-1} \left(\frac{1'8''}{3'6''}\right)$

$\theta = 25.42°$

$R_1 = \frac{F}{2 \cos \theta}$

$R_1 = \frac{11664}{2 \cos \theta}$

$R_1 = 6460 \text{ lb}$

$F_2 = R_1 \sin \theta = 6460 \sin \theta$

$F_2 = 2780 \text{ lb}$

Design of Frame E-E'

Uniform load on E'  = 1701 lb/ft
Uniform load on E   = 3402 lb/ft
Uniform load on $E_u$ = 5922 lb/ft
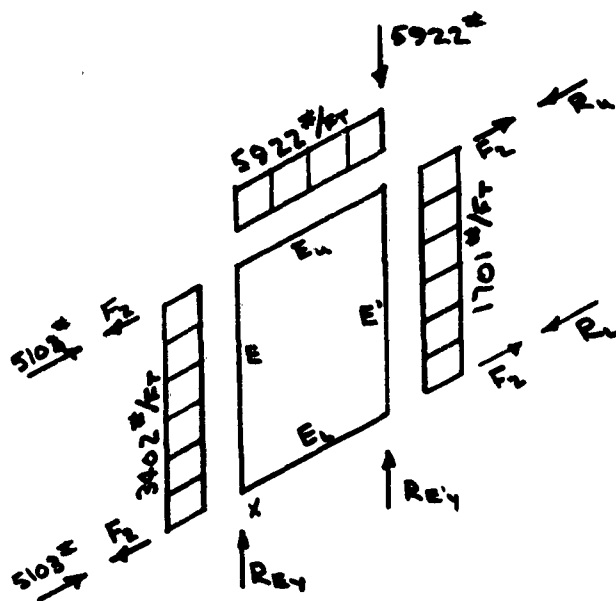
Assume $\dfrac{I_e}{I_{e_u}} = 1$

Assume $\dfrac{I_{e'}}{I_{e_u}} = \dfrac{3}{4} = .75$

$K_e = \dfrac{1}{6}\,\dfrac{3}{4} = \dfrac{1}{8} = .125$ $\qquad\qquad \dfrac{.125}{.375} = .33$

$K_{e_u} = \dfrac{1}{4} = .25$ $\qquad \dfrac{.25}{.344} = .73$ $\qquad\qquad \dfrac{.25}{.375} = .67$

$K_{e'} = \dfrac{1}{6}\,\dfrac{3}{4}\,\dfrac{3}{4} = .094$ $\quad \dfrac{.094}{.344} = .27$

Moment Distribution

| 0 | .33 | .67 | | .73 | .27 | |
|---|---|---|---|---|---|---|
| +10.206 | -10.206 | + 7.896 | -7.896 | +5.103 | | - 5.103 |
| -10.206 | - 5.103 | | | +2.551 | | + 5.103 |
| | + 2.471 | + 4.942 | +2.471 | | | |
| | | - .813 | -1.627 | - .602 | | |
| | + .271 | + .542 | + .271 | | | |
| | | - .099 | - .198 | - .073 | | |
| | + .033 | + .066 | + .033 | | | |
| | | - .012 | - .023 | - .010 | | |
| | + .004 | + .008 | + .004 | | | |
| | | - .001 | - .003 | - .001 | | |
| 0 | -12.530 | +12.529 | -6.968 | +6.968 | | 0 |

282

Calculations of unknowns on Beam $E_u$



$$5922^{\#}/\text{FT}$$
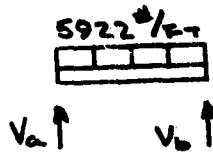
$V_a = V_b = \frac{1}{2}(4)(5922)$

$V_a = V_b = 11{,}844 \text{ lb}$
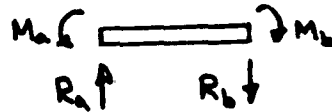
$R_a = R_b = \frac{1}{4}(M_a + M_b)$

$R_a = R_b = \frac{1}{4}(12530 - 6968)$

$R_a = R_b = 1391 \text{ lb}$

$V_A = V_a + R_a =$
$\qquad 11844 + 1391$

$V_A = 13{,}235 \text{ lb}$

$V_B = V_b - R_b =$
$\qquad 11{,}844 - 1391$

$V_B = 10{,}453 \text{ lb}$

$\underline{M_{max.} = M_a = 12{,}530 \text{ ft-lb}}$

SHEAR

$2.24'$

MOMENT

$2293 \text{ FT}^{\#}$
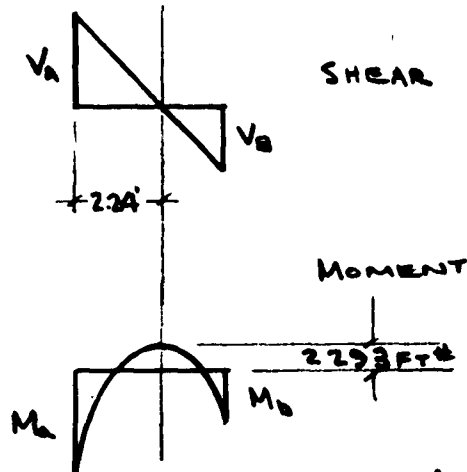
Calculation of unknowns on Beam E

$\curvearrowright$
$\Sigma M = 0 = 5103(6) + 3402(\frac{1}{2})(6)^2$

$\qquad - 6H - 6F_2 - M$

$H = \frac{1}{6}[30{,}618 + 61{,}236 - 16{,}680 - 12{,}530]$

$\underline{H = 10{,}441 \text{ lb}}$

$5103^{\#}$   $M$   $H_1$   $F_2$

$3402^{\#}/\text{FT}$

$5103^{\#}$   $H$   $F_2$

$R_{E_4}$

SHEAR

$12{,}294^{\#}$

$3.61'$

$8119^{\#}$
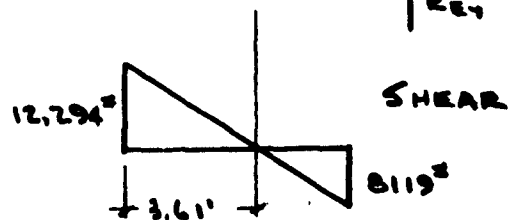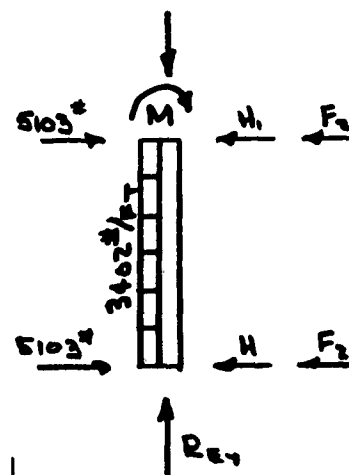
$\Sigma F = F_2 + H_1 + H - 2(5103) - (3402)(6) = 0$

$\qquad H_1 = 10206 + 20412 - 10441 - 5560$

$\qquad \underline{H_1 = 14,617 \text{ lb}}$

$\underline{M = M_{max.} = 12,530 \text{ ft-lb}}$



MOMENT

9660 FT.#

M
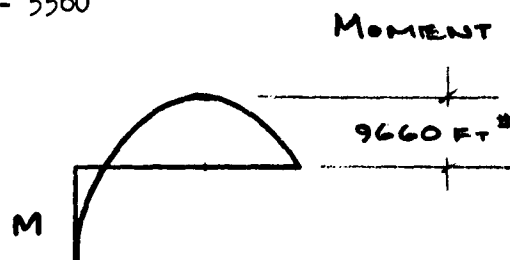
Calculation of unknowns on Beam E'

$\Sigma F_y = R_{E'_y} - V_B - 5922 = 0$

$\qquad R_{E'_y} = 10,453 + 5922$

$\qquad \underline{R_{E'_y} = 16,375 \text{ lb}}$

Refer to free body diagram of
entire frame, page 247, and
solve for $R_u$ and $R_L$



$\overset{+}{\Sigma}M_x = R_{E'_y} \, 4 + \frac{1}{2}(6)^2(1701) + 6R_u + 6F_2$

$\qquad - 6(5103) - \frac{1}{2}(6)^2(3402) - \frac{1}{2}(4)^2(5922)$

$\qquad - 4(5922) - 6F_2 = 0$

$\qquad R_u = \frac{1}{6}[-30,618 - 65,500 + 30,618 + 61,236$

$\qquad\qquad + 47,376 + 23,688]$

$\qquad \underline{R_u = 11,133 \text{ lb}}$
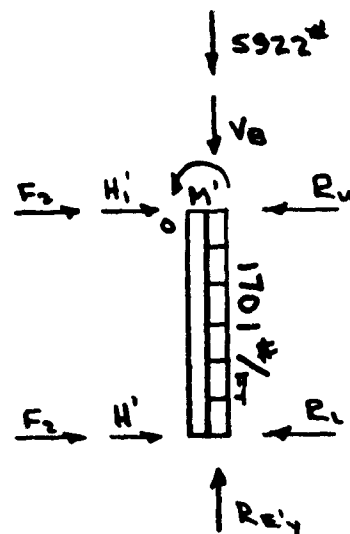
$\Sigma F = 2(5103) + 6(3402) - R_u - R_L - 1701(6) = 0$

$R_L = 10,206 + 20,412 - 11,133 - 10,206$

$\underline{R_L = 9279 \text{ lb}}$

$\Sigma M_O = M' + 6H' + 6F_2 - 6R_L - \frac{1}{2}(6)^2 \, 1701 = 0$

$H' = \frac{1}{6}[55,674 + 30,618 - 16,680 - 6968]$

$\underline{H' = 10,441 \text{ lb}}$

$\Sigma F = 0 = 2F_2 + H' + H'_1 - 6(1701) - R_u - R_L$

$H'_1 = 10,206 + 11,133 + 9279 - 10,441 - 5560$

$\underline{H'_1 = 14,617 \text{ lb}}$

$\underline{M_{max.} = M' = 6,968 \text{ ft-lb}}$

SHEAR.

6264#     3942#

3.68'

MOMENT

4557 FT#

M'
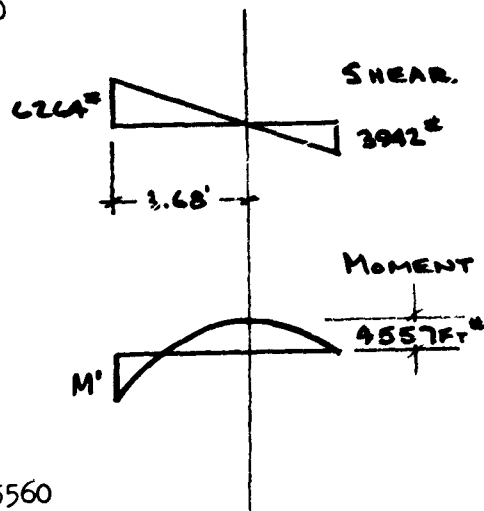
Design of Beam $E_u$

    Maximum bending moment      $M = 12,530 \text{ ft-lb}$
    Axial compressive load      $H'_1 = 14,617 \text{ lb}$

$$\frac{1}{50,000}\left[\frac{14,617}{A} + \frac{12,530(12)}{S_{xx}}\right] \leq 1$$

$$\frac{.2923}{A} + \frac{3.007}{S_{xx}} \leq 1$$

Try 4 WF 10     $A = 2.93 \text{ in.}^2$     $S_{xx} = 4.16 \text{ in.}^3$     $I_{xx} = 8.31 \text{ in.}^4$

$$\frac{.2923}{2.93} + \frac{3.007}{4.16} = .100 + .721 = .821 < 1$$

Use 4 WF 10

Design of Beam E

Maximum bending moment      $M = 12{,}530$
Axial compressive load      $R_{E_y} = 13{,}235$ lb

$$\frac{1}{50{,}000}\left[\frac{13{,}235}{A} + \frac{12{,}530(12)}{S_{xx}}\right] \leq 1$$

$$\frac{.265}{A} + \frac{3.007}{S_{xx}} \leq 1$$

Try 4 WF 10      $A = 2.93$ in.$^2$      $S_{xx} = 4.16$ in.$^3$      $I_{xx} = 8.31$ in.$^4$

$$\frac{.265}{2.93} + \frac{3.007}{4.16} = .091 + .723 = .814 < 1$$

Ratio of $\dfrac{I_E}{I_{E_u}} = 1$    as assumed

Use 4 WF 10

Design of Beam E'

Maximum bending moment      $M' = 6968$ ft-lb
Axial compressive load      $R_{E'_y} = 16{,}375$ lb

$$\frac{1}{50{,}000}\left[\frac{16375}{A} + \frac{6968(12)}{S_{xx}}\right] \leq 1$$

$$\frac{.3275}{A} + \frac{1.672}{S_{xx}} \leq 1$$

Try 4 I 7.7      $A = 2.21$ in.$^2$      $S_{xx} = 3.0$ in.$^3$      $I_{xx} = 6$ in.$^4$

$$\frac{.3275}{2.21} + \frac{1.672}{3.0} = .148 + .557 = .705 < 1$$

Ratio of $\dfrac{I_{E'}}{I_{E_u}} = \dfrac{6}{8.31} = .721 = .75$ as assumed

Use 4 I 7.7

Design of Beam $E_L$

Axial compressive load $\qquad$ H = 10,441 lb

$$A = \frac{10,441}{50,000} = .209 \text{ in.}^2$$

Use L 1 x 1 x 1/8 $\qquad$ A = .23 in.$^2$ $\qquad$ .80 lb/ft

Design of Beams HE and HE'

Axial load $\qquad$ $R_1 = 6460$

$$A = \frac{6460}{50,000} = .1292 \text{ in.}^2$$

Use L 1 x 1 x 1/8 $\qquad$ A = .23 in.$^2$ $\qquad$ .80 lb/ft

Design of Frame F-$F_u$
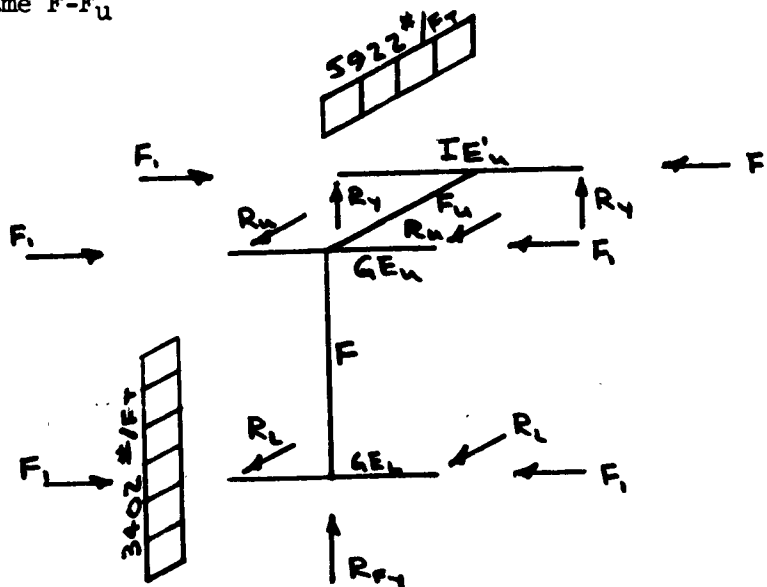


$R_L = R_u = \frac{1}{4}(3402)(6) = \underline{5103 \text{ lb}}$

$R_y = \frac{1}{4}(4)(5922) = \underline{5922 \text{ lb}}$

$R_{F_y} = \frac{1}{2}(4)(5922) = \underline{11,844 \text{ lb}}$

All beams simply connected.

Calculation of unknowns on Beam F

$$M_{max.} = \frac{wL^2}{8} - \left(\frac{2}{12}\right)(R_E) = \frac{3402(6)^2}{8} - \frac{1}{6}(11844)$$

$$\underline{M_{max.} = 13,332 \text{ ft-lb}}$$
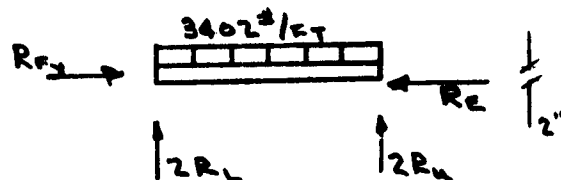
Calculation of unknowns on Beams $GE_u$ and $GE_L$

$$M_{max.} = \frac{2R_u L}{4} = \frac{2(5103)(4)}{4}$$

$$\underline{M_{max.} = 10,206 \text{ ft-lb}}$$
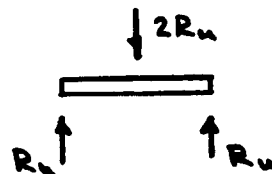
Calculation of unknowns on Beam $F_u$

$$M_{max.} = \frac{wL^2}{8} = \frac{5922(16)}{8}$$
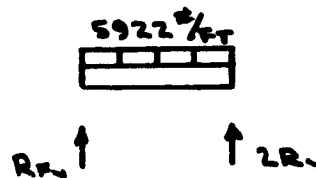
$$\underline{M_{max.} = 11,844 \text{ ft-lb}}$$

Calculation of unknowns on Beam $IE_u'$

$$M_{max.} = \frac{2R_y L}{4} = \frac{2(5922)(4)}{4}$$
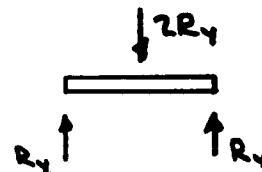
$$\underline{M_{max.} = 11,844 \text{ ft-lb}}$$

Design of Beam F

| Maximum bending moment | M | = 13,332 ft-lb |
|---|---|---|
| Axial compressive load | $R_{F_y}$ | = 11,844 lb |

$$\frac{1}{50,000}\left[\frac{11844}{A} + \frac{13,332(12)}{S_{xx}}\right] \leq 1$$

$$\frac{.237}{A} + \frac{3.2}{S_{xx}} \leq 1$$

Try 4 WF 10    A = 2.93 in.$^2$    $S_{xx}$ = 4.16 in.$^3$    $I_{xx}$ = 8.31 in.$^4$

$$\frac{.237}{2.93} + \frac{3.2}{4.16} = .081 + .770 = .851 < 1$$

Use 4 WF 10

Design of Beams GE$_u$ and GE$_L$

Maximum bending moment    $M_{max.}$ = 10,206 ft-lb
Axial compressive load    $F_1$ = 5,832 lb

$$\frac{1}{50,000}\left[\frac{5832}{A} + \frac{10,206(12)}{S_{xx}}\right] \leq 1$$

$$\frac{.117}{A} + \frac{2.45}{S_{xx}} \leq 1$$

Try 4 I 7.7    A = 2.21 in.$^2$    $S_{xx}$ = 3.0 in.$^3$

$$\frac{.117}{2.21} + \frac{2.45}{3.0} = .053 + .817 = .870 < 1$$

Use 4 I 7.7

Design of Beams F$_u$

Maximum bending moment    $M_{max.}$ = 11,844 ft-lb
Axial load = 0

$$\frac{1}{50,000} \frac{(11844)(12)}{S_{xx}} \leq 1$$

$$\frac{2.842}{S_{xx}} \leq 1$$

Use 4 I 7.7    $S_{xx}$ = 3.0 in.$^3$

Design of Beam IE$_u'$

Maximum bending moment    $M_{max.}$ = 11,844 ft-lb
Axial compressive load    $F_1$ = 5,832 lb

$$\frac{1}{50,000} \left[ \frac{5832}{A} + \frac{11,844(12)}{S_{xx}} \right] \leq 1$$

$$\frac{.1166}{A} + \frac{2.842}{S_{xx}} \leq 1$$

Try 4 I 7.7     A = 2.21 in.$^2$     $S_{xx}$ = 3.0 in.$^3$

$$\frac{.1166}{2.21} + \frac{2.842}{3.0} = .053 + .947 = 1.000 = 1$$

Use 4 I 7.7

Design of Frame G-H-I
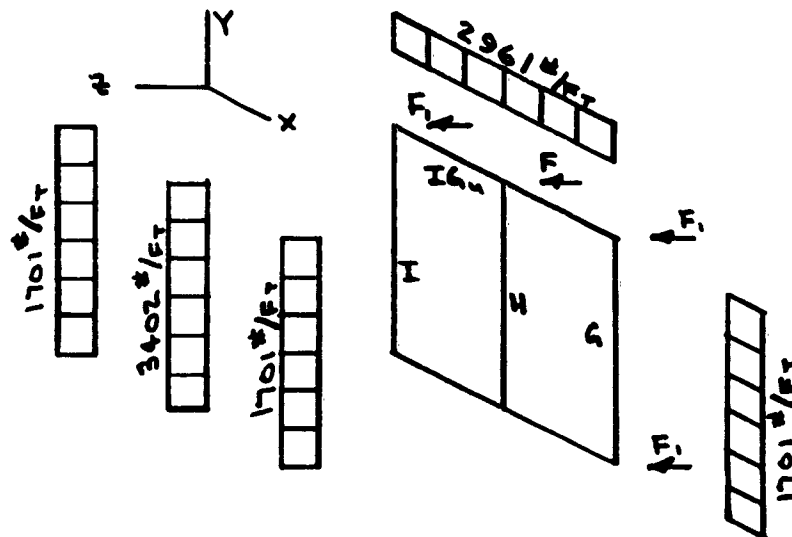


Beam IG$_u$ is simply connected to Beams I and G

Calculation of unknowns on Beam G in Y-Z plane

$F_1 = \frac{1}{2}(1701)(6)$

$F_1$ = 5103 lb

$M_{max.} = \frac{wL^2}{8} = \frac{1701(6)^2}{8}$

$M_{max.}$ = 7655 ft-lb
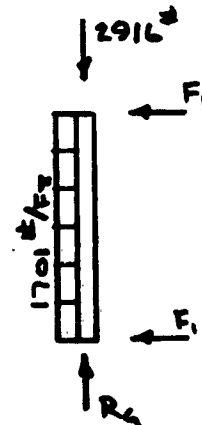
290

Calculation of unknowns on Beam G in X-Y plane

$H = \frac{1}{2}(1701)(6) = 5103$ lb

$M_{max.} = \frac{wL^2}{8} = \frac{1701(6)^2}{8}$

$\underline{M_{max.} = 7655 \text{ ft-lb}}$
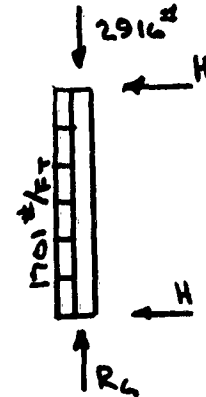
Calculation of unknowns on Beam H

$F = \frac{1}{2}(3402)(6) = 10,206$ lb

$M_{max.} = \frac{wL^2}{8} = \frac{(3402)(6)^2}{8}$

$\underline{M_{max.} = 15,309 \text{ ft-lb}}$
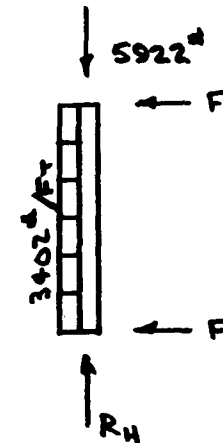
Calculation of unknowns on Beam I

$R_I = 2961 + 5922 = 8883$ lb

$M_{max.} = \frac{wL^2}{8} - \frac{2}{12}(5922)$

$\underline{M_{max.} = 6685 \text{ ft-lb}}$
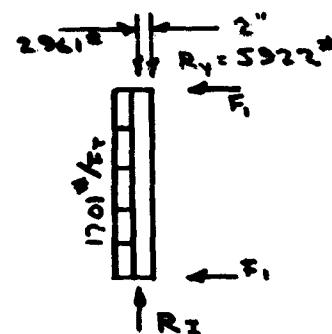
Calculation of unknowns on Beam $IG_u$

<u>$M_{max.}$ = 1481 ft-lb</u>

R = 5103 + 5103 lb = 10,206 lb
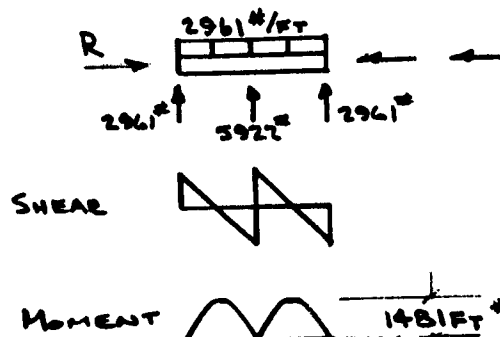


Design of Beam G

Maximum bending moment in X-Y plane    $M_{max.}$ = 7655 ft-lb
Maximum bending moment in Y-Z plane    $M_{max.}$ = 7655 ft-lb
Axial compressive load    $R_G$ = 2916 lb

$$\frac{f_a}{F_a} + \left(\frac{f_b}{F_b}\right)_{xx} + \left(\frac{f_b}{F_b}\right)_{yy} \leq 1$$

$$\frac{1}{50,000}\left[\frac{2916}{A} + \frac{7655(12)}{S_{xx}} + \frac{7655(12)}{S_{yy}}\right] \leq 1$$

$$\frac{.058}{A} + \frac{1.837}{S_{xx}} + \frac{1.837}{S_{yy}} \leq 1$$

Try 5 WF 16    A = 4.70 in.$^2$    $S_{xx}$ = 8.53 in.$^3$    $S_{yy}$ = 3.00 in.$^3$

$$\frac{.058}{4.70} + \frac{1.837}{8.53} + \frac{1.837}{3.00} = .014 + .216 + .613 = .843 < 1$$

<u>Use 5 WF 16</u>
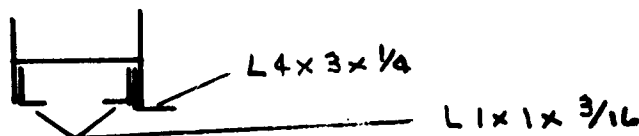


Design of Beam H

$M_{max.}$ = 15,309 ft-lb

$R_H$ = 5,922 lb

$$\frac{1}{50,000}\left[\frac{5922}{A} + \frac{15309(12)}{S_{xx}}\right] \leq 1$$

$$\frac{.1184}{A} + \frac{3.674}{S_{xx}} \leq 1$$

Try 4 WF 10    A = 2.93 in.$^2$    $S_{xx}$ = 4.16 in.$^3$

$$\frac{.1184}{2.93} + \frac{3.674}{4.16} = .041 + .883 = .924 < 1$$

Use 4 WF 10

Design of Beam I

$M_{max.}$ = 6685 ft-lb

$R_I$ = 8883 lb

$$\frac{1}{50,000}\left[\frac{8883}{A} + \frac{6685(12)}{S_{xx}}\right] \leq 1$$

$$\frac{.1777}{A} + \frac{1.604}{S_{xx}} \leq 1$$

Try 4 ⌐ 5.4    A = 1.56 in.$^2$    $S_{xx}$ = 1.9 in.$^3$

$$\frac{.1777}{1.56} + \frac{1.604}{1.9} = .114 + .845 = .959 < 1$$

Use 4 I 7.7

Design of Beam IG$_u$

$M_{max.}$ = 1481 ft-lb

R    = 10,206 lb

$$\frac{1}{50,000}\left[\frac{10206}{A} + \frac{1481(12)}{S_{xx}}\right] \leq 1$$

$$\frac{.2041}{A} + \frac{.3554}{S_{xx}} \leq 1$$

Try L 4 x 4 x 1/4    A = 1.94 in.$^2$    $S_{xx}$ = 1.1 in.$^3$

$$\frac{.2041}{1.94} + \frac{.3554}{1.1} = .105 + .323 = .428 < 1$$

Use L 4 x 4 x 1/4

Design of Struts

   Axial load         R = 10,206 lb

$$A = \frac{10,206}{50,000} = .204 \text{ in.}^2$$

   Use L 1 x 1 x 1/8     A = .23 in.$^2$

Loading on Frame Q-Q', R-R'
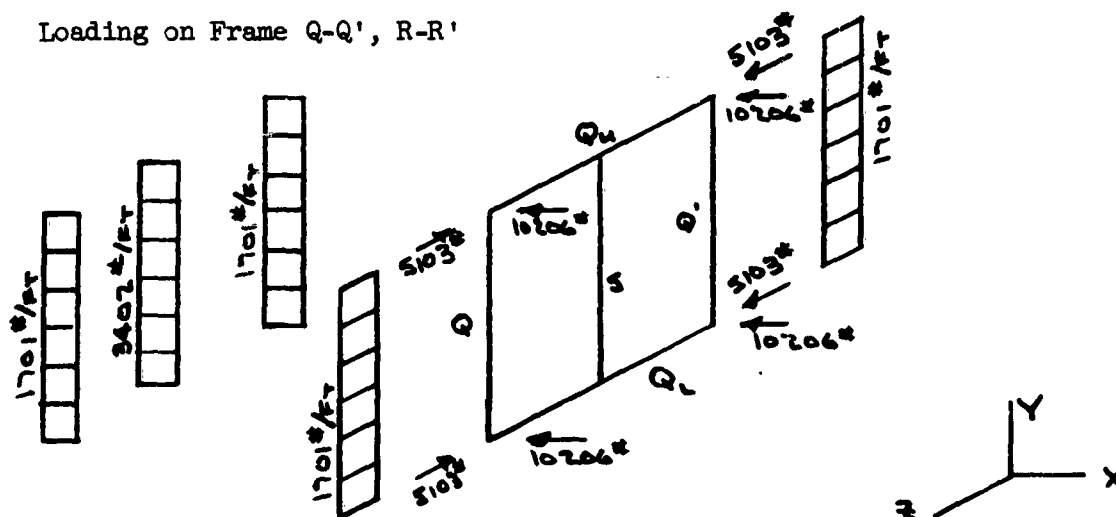


   All beams are simply connected.

Calculation of unknowns on Beams Q and Q', R and R', in X-Y plane

$$M_{max.} = \frac{wL^2}{8} = \frac{1701(6)^2}{8}$$

$$M_{max.} = 7655 \text{ ft-lb}$$
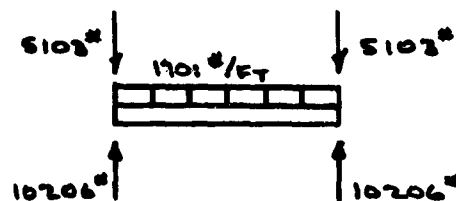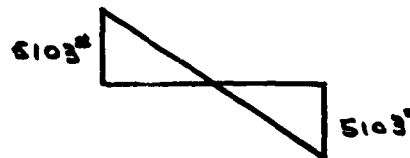
Calculation of unknowns on Beams Q and Q', R and R' in Y-Z plane

$$M_{max.} = \frac{wL^2}{8} = \frac{1701(6)^2}{8}$$

$$\underline{M_{max.} = 7655 \text{ ft-lb}}$$
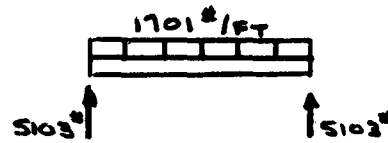
Calculation of unknowns on Beams $Q_u$ and $Q_L$, $R_u$ and $R_L$

$$M_{max.} = \frac{PL}{4} - \frac{2}{12}(5103)$$

$$\underline{M_{max.} = 9355 \text{ ft-lb}}$$
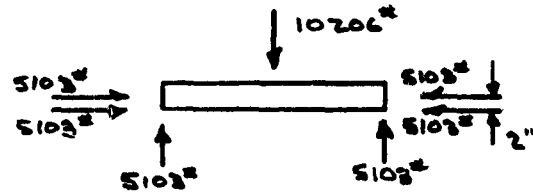
Calculation of unknowns on Beam S, S', T, T'

$$M_{max.} = \frac{wL^2}{8} = \frac{3402(6)^2}{8}$$

$$\underline{M_{max.} = 15,309 \text{ ft-lb}}$$
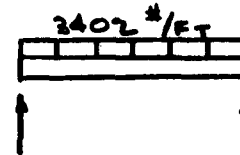
Calculation of unknowns on Beams $Q'R_u'$ and $Q'R_L'$

$$M_{max.} = \frac{PL}{4} = \frac{10,206(4)}{4}$$

$$\underline{M_{max.} = 10,206 \text{ ft-lb}}$$
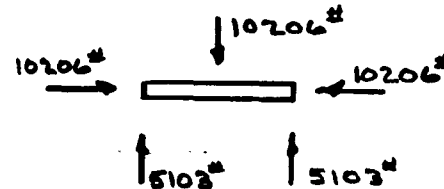
Design of Beams Q and Q', R and R'

$$M_{max.x-y} = 7655 \text{ ft-lb}$$

$$M_{max.y-z} = 7655 \text{ ft-lb}$$

Axial Load = 0

$$\frac{1}{50,000}\left[\frac{0}{A} + \frac{7655(12)}{S_{xx}} + \frac{7655(12)}{S_{yy}}\right] \leq 1$$

$$\frac{1.837}{S_{xx}} + \frac{1.837}{S_{yy}} \leq 1$$

Try 5 WF 16     A = 4.70 in.$^2$     $S_{xx}$ = 8.53 in.$^3$     $S_{yy}$ = 3.00 in.$^3$

$$\frac{1.837}{8.53} + \frac{1.837}{3.00} = .215 + .610 = .825 < 1$$

<u>Use 5 WF 16</u>

Design of Beams $Q_u$ and $Q_L$, $R_u$ and RL

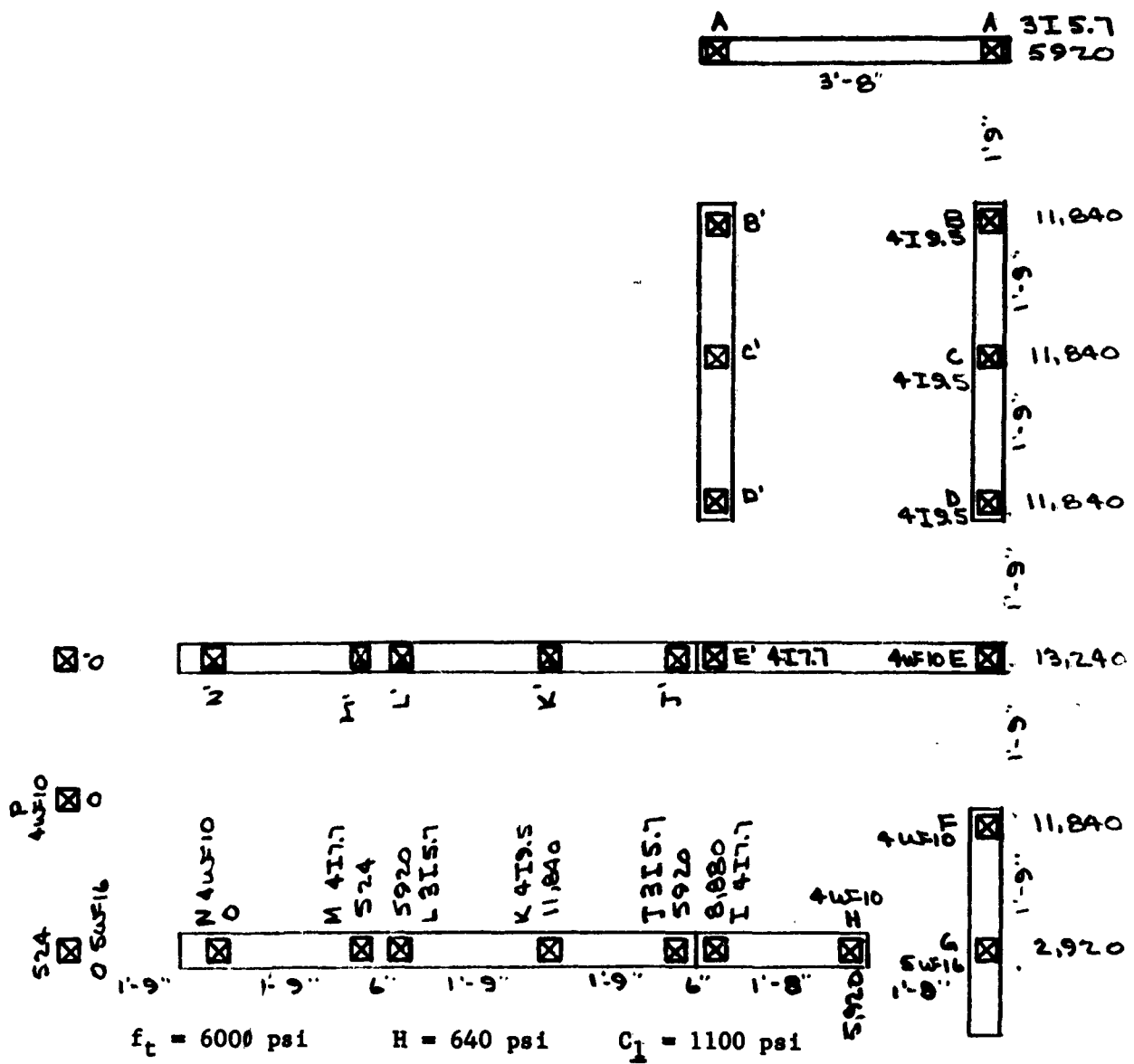$M_{max.}$ = 9355 ft-lb
Axial load = 10,206 lb

$$\frac{1}{50,000} \left[ \frac{10206 \text{ lb}}{A} + \frac{9355(12)}{S_{xx}} \right] \leq 1$$

$$\frac{.2041}{A} + \frac{2.245}{S_{xx}} \leq 1$$

Try 4 I 7.7     A = 2.21 in.$^2$     $S_{xx}$ = 3.0 in.$^3$

$$\frac{.2041}{2.21} + \frac{2.245}{3.0} = .093 + .748 = .841 < 1$$

<u>Use 5 I 10</u>

Design of Beam S, S', T, T'

$M_{max.}$ = 15,309 ft-lb
Axial load = 0

$$\frac{1}{50,000} \left[ \frac{0}{A} + \frac{(15309)12}{S_{xx}} \right] \leq 1$$

$$\frac{3.675}{S_{xx}} \leq 1$$

<u>Use 4 WF 10</u>

Design of Beams $Q'R_u$ and $Q'R_L'$, $QR_u$ and $QR_L$

$M_{max.}$ = 10,206 ft-lb
Axial load = 10,206 lb

$$\frac{1}{50,000} \left[ \frac{10206}{A} + \frac{10206(12)}{S_{xx}} \right] \leq 1$$

$$\frac{.2041}{A} + \frac{2.45}{S_{xx}} \leq 1$$

Try 4 I 7.7     $A = 2.21$ in.$^2$     $S_{xx} = 3.0$ in.$^3$

$$\frac{.2041}{2.21} + \frac{2.45}{3.0} = .093 + .816 = .909 < 1$$

Use 5 I 10

## Total Weight of Structure

Covering = $(12 \text{ psf})(4)(4')(6') = 1,152$ lb

Steel Members

| | | | |
|---|---|---|---|
| R', R, Q, Q' | $= 4(6')(16 \text{ lb/ft}) = 384$ lb | 5 WF 16 |
| S, S', T, T' | $= 4(6')(10 \text{ lb/ft}) = 240$ lb | 4 WF 10 |
| Q'R$_u$', Q'R$_L$', QR$_u$, QR$_L$ | $= 4(4')(10 \text{ lb/ft}) = 160$ lb | 5 I 10 |
| Q$_u$, Q$_L$, R$_u$, R$_L$ | $= 4(4')(10 \text{ lb/ft}) = \underline{160 \text{ lb}}$ | 5 I 10 |
| | Total | 2096 lb |

This load will be distributed between four supporting columns.

$$F_{C} = \frac{1}{4}(2096) = 524 \text{ lb}$$

Load on Frame 0-0'
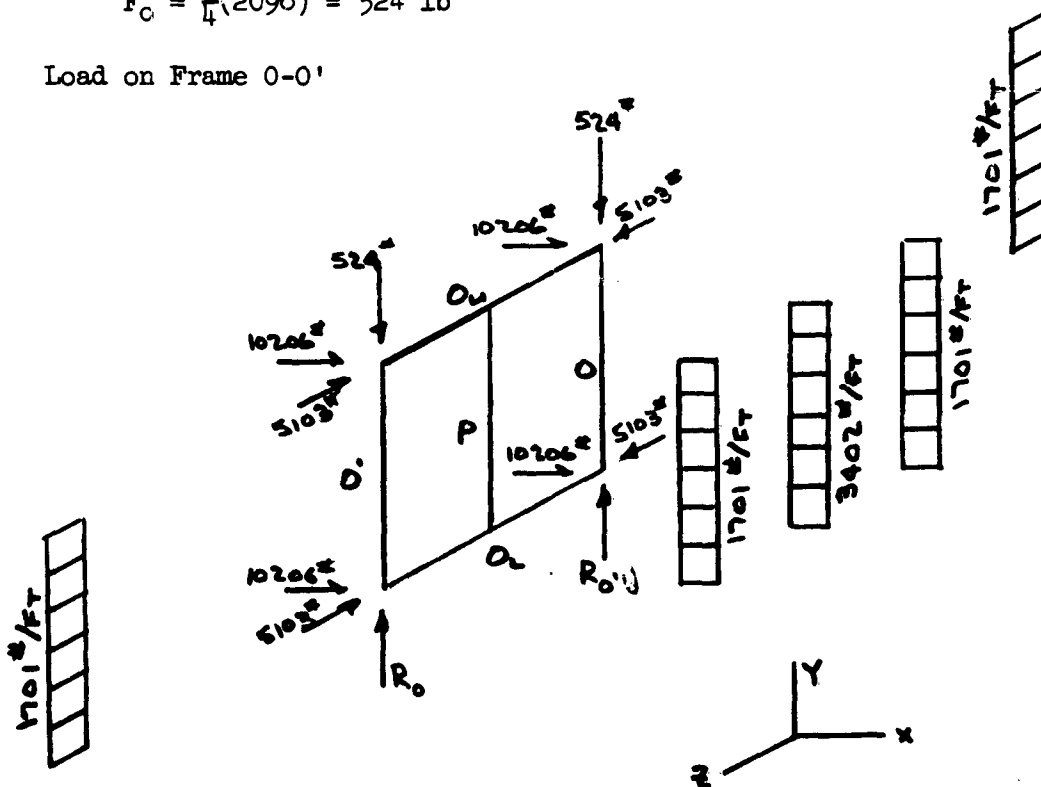
Calculation of unknowns on Beams O and O' in X-Y plane

$$M_{max.} = \frac{wL^2}{8} = \frac{1701(6)^2}{8}$$

$$\underline{M_{max.} = 7655 \text{ ft-lb}}$$
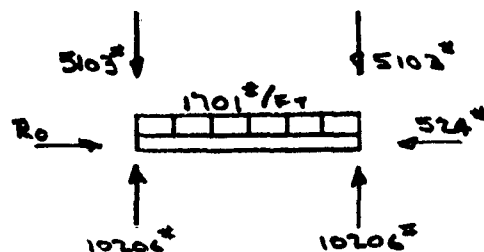
Calculation of unknowns on Beams O and O' in Y-Z plane

$$M_{max.} = \frac{wL^2}{8} = \frac{1701(6)^2}{8}$$

$$\underline{M_{max.} = 7655 \text{ ft-lb}}$$
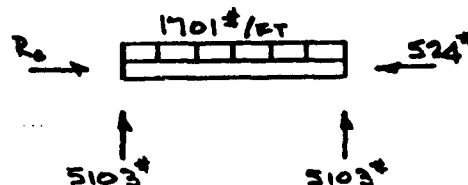
Calculation of unknowns on Beams P, N, N'

$$M_{max.} = \frac{wL^2}{8} = \frac{3402(6)^2}{8}$$

$$\underline{M_{max.} = 15,309 \text{ ft-lb}}$$
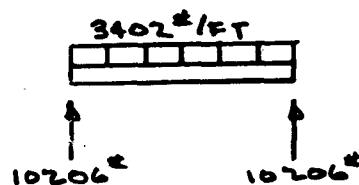
Calculation of unknowns on Beams $O_u$ and $O_L$

$$M_{max.} = \frac{PL}{4} - \frac{2}{12}(5103)$$

$$\underline{M_{max.} = 9355 \text{ ft-lb}}$$
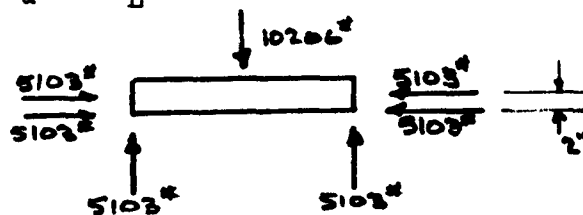
Calculation of unknowns on Beams M and M'

$$M_{max.} = \frac{wL^2}{8} = \frac{1701(6)^2}{8}$$
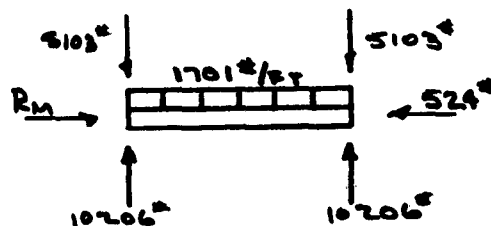
$$\underline{M_{max.} = 7655 \text{ ft-lb}}$$

Design of Beams $M_u$ and $M_L$

$$A = \frac{P}{S} = \frac{10,206}{50,000} = .2041 \text{ in.}^2$$

Use L 1x1x1/8   A = .23 in.$^2$   .80 lb/ft

Calculation of unknowns on Beams $OM_u$ and $OM_L$, $O'M_u'$ and $O'M_L'$

$$M_{max.} = \frac{PL}{4} = \frac{10,206(4)}{4}$$

$$\underline{M_{max.} = 10,206 \text{ ft-lb}}$$
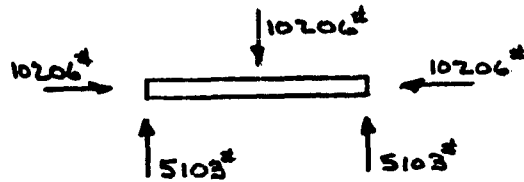
Design of Beam O and O'

$$M_{max.x-y} = 7655 \text{ ft-lb}$$

$$M_{max.y-z} = 7655 \text{ ft-lb}$$

$$R_0 = 524 \text{ lb}$$

$$\frac{1}{50,000}\left[\frac{524}{A} + \frac{7655(12)}{S_{xx}} + \frac{7655(12)}{S_{yy}}\right] \leq 1$$

$$\frac{.0105}{A} + \frac{1.837}{S_{xx}} + \frac{1.837}{S_{yy}} \leq 1$$

Try 5 WF 16   A = 4.70 in.$^2$   $S_{xx}$ = 8.53 in.$^3$   $S_{yy}$ = 3.00 in.$^3$

$$\frac{.0105}{4.70} + \frac{1.837}{8.53} + \frac{1.837}{3.00} = .002 + .215 + .610 = .827 < 1$$

<u>Use 5 WF 16</u>

Design of Beams P, N, N'

Since the loading is the same as that for beams S, S', T, T', the same members can be used.

<u>Use 4 WF 10</u>

Design of Beams $O_u$ and $O_L$

Since the loading is the same as that for beams $Q_u$ and $Q_L$, $R_u$ and $R_L$, the same member can be used.

Use 5 I 10

Design of Beams M and M'

$M_{max.}$ = 7655 ft-lb
$R_M$ = 524 lb

$$\frac{1}{50,000}\left[\frac{524}{A} + \frac{7655(12)}{S_{xx}}\right] \leq 1$$

$$\frac{.0105}{A} + \frac{1.837}{S_{xx}} \leq 1$$

Try 4 I 7.7      A = 2.21 in.$^2$      $S_{xx}$ = 3.00 in.$^3$

$$\frac{.0105}{2.21} + \frac{1.837}{3.00} = .005 + .612 = .617 < 1$$

Use 4 I 7.7

Design of Beams $OM_u$ and $OM_L$, $O'M'_u$ and $O'M'_L$

$M_{max.}$ = 10206 ft-lb
Axial load = 10206 lb

$$\frac{1}{50,000}\left[\frac{10206}{A} + \frac{10206(12)}{S_{xx}}\right] \leq 1$$

$$\frac{.2041}{A} + \frac{2.45}{S_{xx}} \leq 1$$

Try 4 I 7.7      A = 2.21 in.$^2$      $S_{xx}$ = 3.0 in.$^3$

$$\frac{.2041}{2.21} + \frac{2.245}{3.0} = .093 + .816 = .909 < 1$$

Use 4 I 7.7

300

Entranceway Foundation



$f_t = 6000$ psi      $H = 640$ psi      $C_1 = 1100$ psi
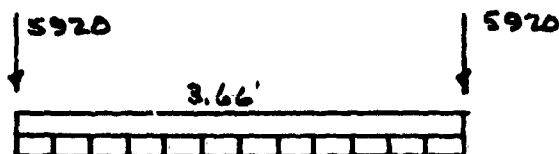
$$w = \frac{11,840}{3.66} = 3240 \text{ lb/ft}$$

$$M_{max.} = \frac{wL}{8} = \frac{11,840 \times 3.66 \times 12}{8} = 65,100 \text{ in.-lb}$$

$$S = \frac{M}{f} = \frac{65,100}{6000} = 10.9 = \frac{bh^2}{6} \qquad \text{Try } b = 11.5\text{"}$$

$$h^2 = \frac{10.9 \times 6}{11.5} = 5.7 \qquad h = 2.39\text{"} \qquad \text{Try } 12\text{"} \times 3\text{"}$$

$$H = \frac{3V}{2bh} = \frac{3 \times 3240 \times 3.16}{2 \times 11.5 \times 2.62} = \frac{30,700}{60.2} = 510 < 640$$

Bearing Plates $\qquad A_{min.} = \frac{5920}{1100} = 5.39 \text{ sq in.} \qquad$ Use 4" x 4" x 1/4"

Use 12" x 3" x 3'-8" sill
2 - 4" x 4" x 1/4" bearing plates

$$P_{B-S} = \frac{11,840}{4 \times \frac{11.5}{12}} = 3090 \text{ psf}$$
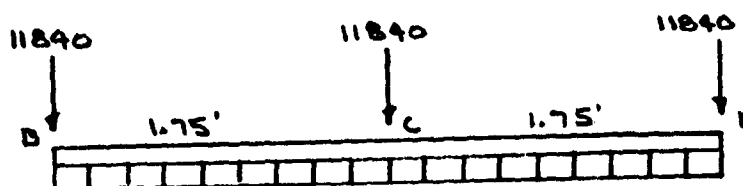


$$w = \frac{35,520}{3.5} = 10,130 \text{ lbs/ft}$$
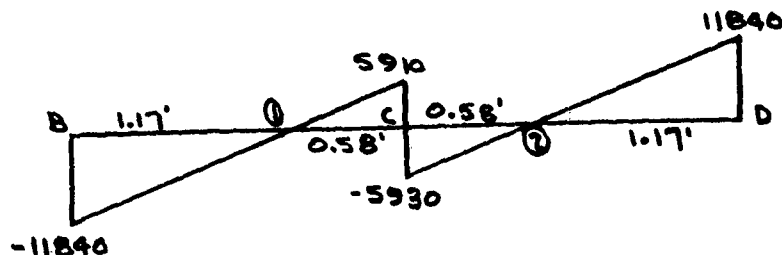
Max. $M = M_1 = \frac{1}{2}$ x 11,840 x 1.17 x 12 = 83,000 in.-lb

$S = \frac{M}{f} = \frac{83,000}{6,000} = 13.8$      Try b = 11.5

$S = 13.8 = \frac{bh^2}{6}$      $h^2 = \frac{13.8 \text{ x } 6}{11.5} = 7.2$      h = 2.69

Try b = 13.5      $h^2 = \frac{13.8 \text{ x } 6}{13.5} = 6.14$      h = 2.48

Try 14" x 3"

$H = \frac{3V}{2bh} = \frac{3 \text{ x } 11,840}{2 \text{ x } 13.5 \text{ x } 2.62} = 503 < 640$

$A_{B-min.} = \frac{11,840}{1100} = 10.8$ sq in.      4" x 5" x 1/4"

Use 14" x 3" x 3'-10" sills (2)
6 - 5" x 4" x 1/4" Bearing plate

$P_{B-S} = \frac{35,520}{3.835 \text{ x } 1.12} = 8,250$ psf
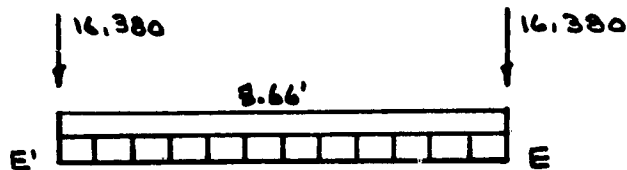


$M = \frac{wL}{8} = \frac{16,380 \text{ x } 2 \text{ x } 3.66 \text{ x } 12}{8} = 165,000$ in.-lb

$S = \frac{M}{f} = \frac{165,000}{6000} = 27.5 = \frac{bh^2}{6}$

$P_{B-S} = 8,000$ psf      $A_s = \frac{33,760}{8000} = 4.21$ sq ft

$b_{min.} = \frac{4.21}{3.66} = 1.15' = 1" - 1.8" = 13"$

Try b = 13.5"

$h^2 = \dfrac{27.5 \times 6}{13.5} = 12.2 \qquad h = 3.5" \qquad$ Try 14" x 4"

$H = \dfrac{3V}{2bh} = \dfrac{3 \times 13,400}{2 \times 13.5 \times 3.62} = 410 < 640$

$A_{B-M} = \dfrac{16,380}{1100} = 14.9$ sq in.　5" x 5" WF　　5" x 4" I

Use:　14" x 4" sill
1 - 5" x 5" x 1/4" Plate (4 WF 10)
1 - 5" x 4" x 1/4" Plate (4 I 7.7)
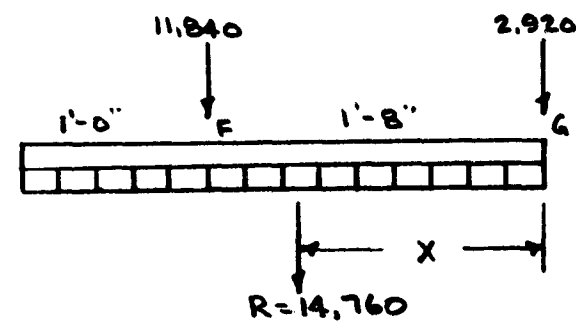


$\Sigma M_G = 0$

$11,840 \times 1.66 = 14,760\,x$

$x = \dfrac{11,840 \times 1.66}{14,760} = 1.33'$

$2x = 2.66' = 2'-8"$

$w = \dfrac{14,760}{2.66} = 5,550$ lb/ft
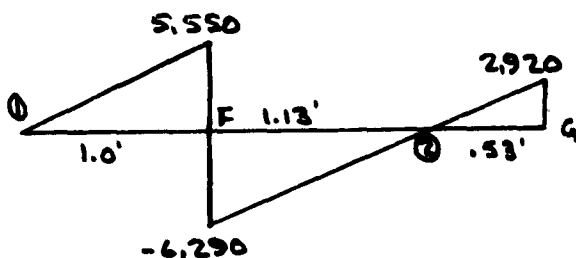
$M_{max.} = M_F = \dfrac{1}{2} \times 1.0 \times 12 \times 5,550 = 33,300$ in.-lb

$S = \dfrac{M}{f} = \dfrac{33,300}{6,000} = 5.55 = \dfrac{bh^2}{6}$

$A_{S-M} = \dfrac{14,760}{8,000} = 1.85$ sq in.

$b_{min.} = \dfrac{1.85}{2.66} \times 12 = 8.35" \qquad$ Try b = 9.5"

$$h^2 = \frac{5.55 \times 6}{9.5} = 3.5 \qquad h = 1.87" \qquad \text{Try } b = 7.5"$$

$$h^2 = \frac{5.55 \times 6}{7.5} = 4.44 \qquad h = 2.11" \qquad \text{Try } 8" \times 3"$$

$$H = \frac{3V}{2bh} = \frac{3 \times 6,290}{2 \times 2.62 \times 7.5} = 480 < 640$$

$$A_{F\text{-min.}} = \frac{11,840}{1100} = 10.8 \text{ sq in.} \qquad A_{G\text{-min.}} = \frac{2,920}{1,100} = 2.66 \text{ sq in.}$$

Use:  8" x 3" x 3'-1" sill
      1 - 6" x 6" x 1/4"  PL  (5 WF 16)
      1 - 5" x 5" x 1/4"  PL  (4 WF 10)

$$P_{SB} = 5550 \times \frac{12}{7.5} = 8900 \text{ psf}$$
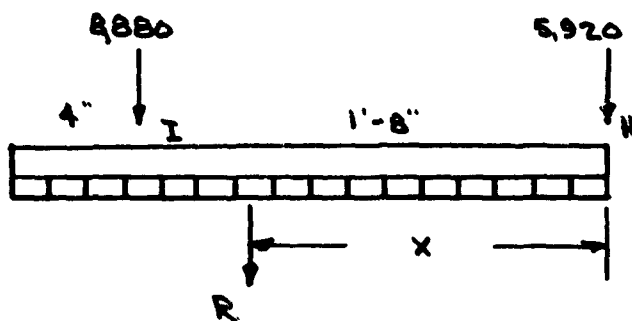


$$R = 8,880 + 5920$$
$$\underline{\phantom{R =}\; 8880}$$
$$14,800$$

$$14,800x = 8,880 \times 1.66$$

$$x = .995' = 11.95"$$

$$w = \frac{14,800}{2 \times .995} = 7,430 \text{ lb/ft}$$
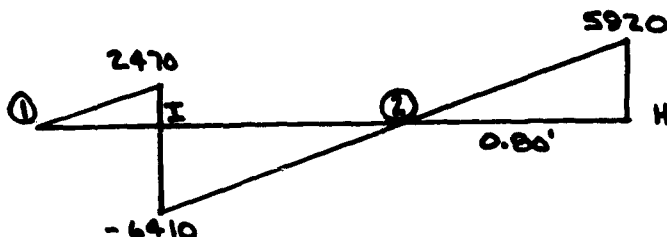


$$M_{max.} = M_2 = \frac{1}{2} \times 0.80 \times 12 \times 5920 = 28,400 \text{ in.-lb}$$

$$S = \frac{M}{f} = \frac{28,400}{6,000} = 4.74 = \frac{bh^2}{6} \qquad \text{Try } b = 5.5$$

$$h^2 = \frac{4.74 \times 6}{5.5} = 5.16 \qquad h = 2.27 \qquad \text{Try } 6" \times 3"$$

$$H = \frac{3V}{2bh} = \frac{3 \times 6410}{2 \times 5.62 \times 2.62} = 655 > 640 \text{ allow.}$$

$$A_{I-M} = \frac{8880}{1100} = 8.06 \text{ sq in.} \qquad A_{H-M} = \frac{5920}{1100} = 5.39 \text{ sq in.}$$

$$\text{Use:} \quad 6" \times 3" \times 2'\text{-}2" \text{ sill}$$
$$\underline{\quad\quad\quad\quad 5" \times 5" \times 1/4" \quad \text{PL} \quad (4 \text{ WF } 10)}$$
$$\underline{\quad\quad\quad\quad 5" \times 4" \times 1/4" \quad \text{PL} \quad (4 \text{ I } 7.7)}$$

$$P_{SB} = \frac{7430}{5.5} \times 12 = 16,200 \text{ psf}$$



```
        5920           11840            5920
         |               |                |
         v               v                v
   L          1.75'      K      1.75'         J
   ─────────────────────────────────────────
   │ │ │ │ │ │ │ │ │ │ │ │ │ │ │ │ │ │ │ │ │
   ─────────────────────────────────────────
               w = 6760 lbs/ft
```
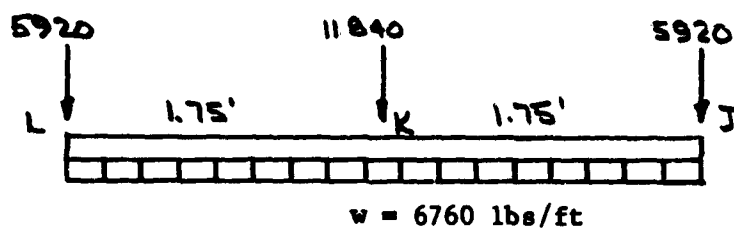
$$\begin{array}{r} 11,840 \\ 5,920 \\ \underline{5,920} \\ 23,680 \end{array}$$



```
                    5920            5920
   .875            /|    .875      /|
          \      /  |          \  / |
   .875     .875   .875 .875
   -5920            -5920
```
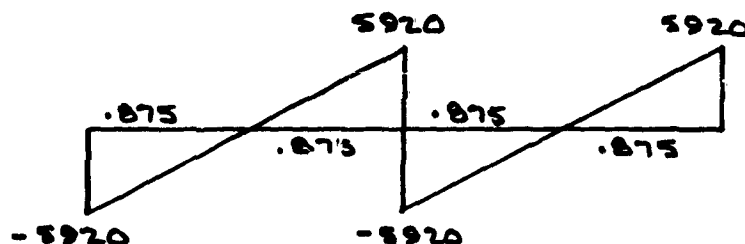
$$\begin{array}{r} 1'\text{-}9" \\ 1'\text{-}9" \\ \underline{4"} \\ 2'\text{-}22" \\ 3'\text{-}10" \end{array}$$

$$M_{max.} = \frac{1}{2} \times 5920 \times .875 = 2590 \text{ lb ft} = 31,100 \text{ in.-lb}$$

$$S = \frac{M}{f} = \frac{31,100}{6000} = 5.19 = \frac{bh^2}{6} \qquad \text{Try } b = 7.5$$

$$h^2 = \frac{5.19 \times 6}{7.5} = 4.15 \qquad h = 2.04" \qquad \text{Try } 8" \times 3"$$

$$H = \frac{3V}{2bh} = \frac{3 \times 5920}{2 \times 7.5 \times 2.62} = 526 < 640$$

$$A_{I-M} = \frac{5920}{1100} = 5.39 \text{ sq in.} \qquad A_{WF-M} = \frac{11,840}{1100} = 10.75 \text{ in.}^2$$

Use:  8" x 3" x 3'-10" sill
       5" x 4" x 1/4"  PL  4 I 9.5
       4" x 4" x 1/4"  PL  3 I 5.7

$$P_{S-B} = \frac{6760}{7.5} \times 12 = 10,800 \text{ psf}$$
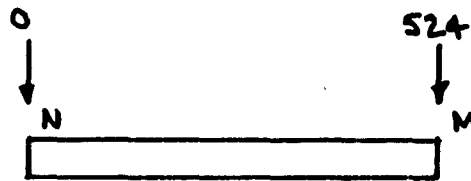


Use 6" x 2" x 2'-1" sill
2 - 4" x 4" x 1/4" PL's

### Blast Cover

Design of Blast Cover

Design Load = 20 psi (5.17')(7.5")(12) = 9,300 lb/plank

Assume Fixed
End Beam
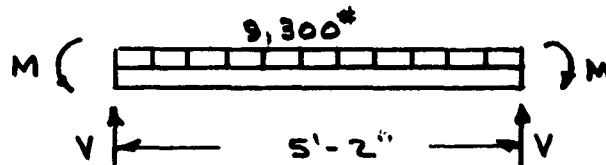


$$V = \tfrac{1}{2}(9,300) = 4650 \text{ lb}$$

$$M = \frac{WL}{12} = \frac{f\, b\, h^2}{6}$$

$$h = \sqrt{\frac{6\,WL}{12\,f\,b}} = \sqrt{\frac{(9300)5.17(12)}{2(6000)(7.5)}}$$

h = 1.86 in.        Try 3" x 8"        h = 2.625

Shear

$$H = \frac{3V}{2bh} = \frac{3(4650)}{2(7.5)(2.625)} = 354 < 640$$

<u>Use 8" x 3" x 5'-0"</u>    wt = 24 lb    bm = 10.0    8 required

Frame

$$W = V = 4650 \text{ lb}$$

$$C_{\perp} = 1100 \text{ psi}$$

$$b = \frac{V}{C_{\perp} L} \qquad L = 7.5"$$

$$b = \frac{4650}{(1100)(7.5)} = .564 \text{ in. minimum}$$

<u>Use 3" x 3" x 5'-0"</u>    wt = 8.5 lb    bm = 3.75    2 required

Footing

$$P_{bear} = 2000 \text{ psf} \qquad \text{Footing load} = 8(9300) = 74,400 \text{ lb}$$

$$\text{Footing Area} = \frac{37,200}{2000} = 18.6 \text{ ft}^2$$

with L = 5.5 ft

$$b = \frac{18.6}{5.5} = 3.38 \text{ ft} \qquad \text{This is much too large.}$$

Since the positive blast phase lasts only 6 sec. for a 20-MT weapon at 20 psi overpressure, we will use a medium soil bearing strength of 8000 psf assuming slow footing settlement.

$$\text{Footing Area} = \frac{37,200}{8000} = 4.65 \text{ ft}^2$$

with    L = 5.5 ft    $b = \frac{4.65}{5.5} = .845 \text{ ft}$

<u>Use 4" x 12" x 5'-6"</u>    wt = 55.6 lb    bm = 22    2 required

Design of Deadman for Blast Cover



5'-0"

8 @ 8" x 8" x 5'-0"

2 @ 3" x 3" x 5'-0"

2 @ 4" x 12" x 5'-6"

5'-6"

SECTION "B-B"

4'-4"

4'-5"

A

A

11,200#

2 5/8"

8 @ 8" x 8" x 5'-0"

4 @ 3" x 3" x 2'-6"

2 @ 4" x 12" x 6.5'

30°

4'-6"

P

b

Section "A-A"
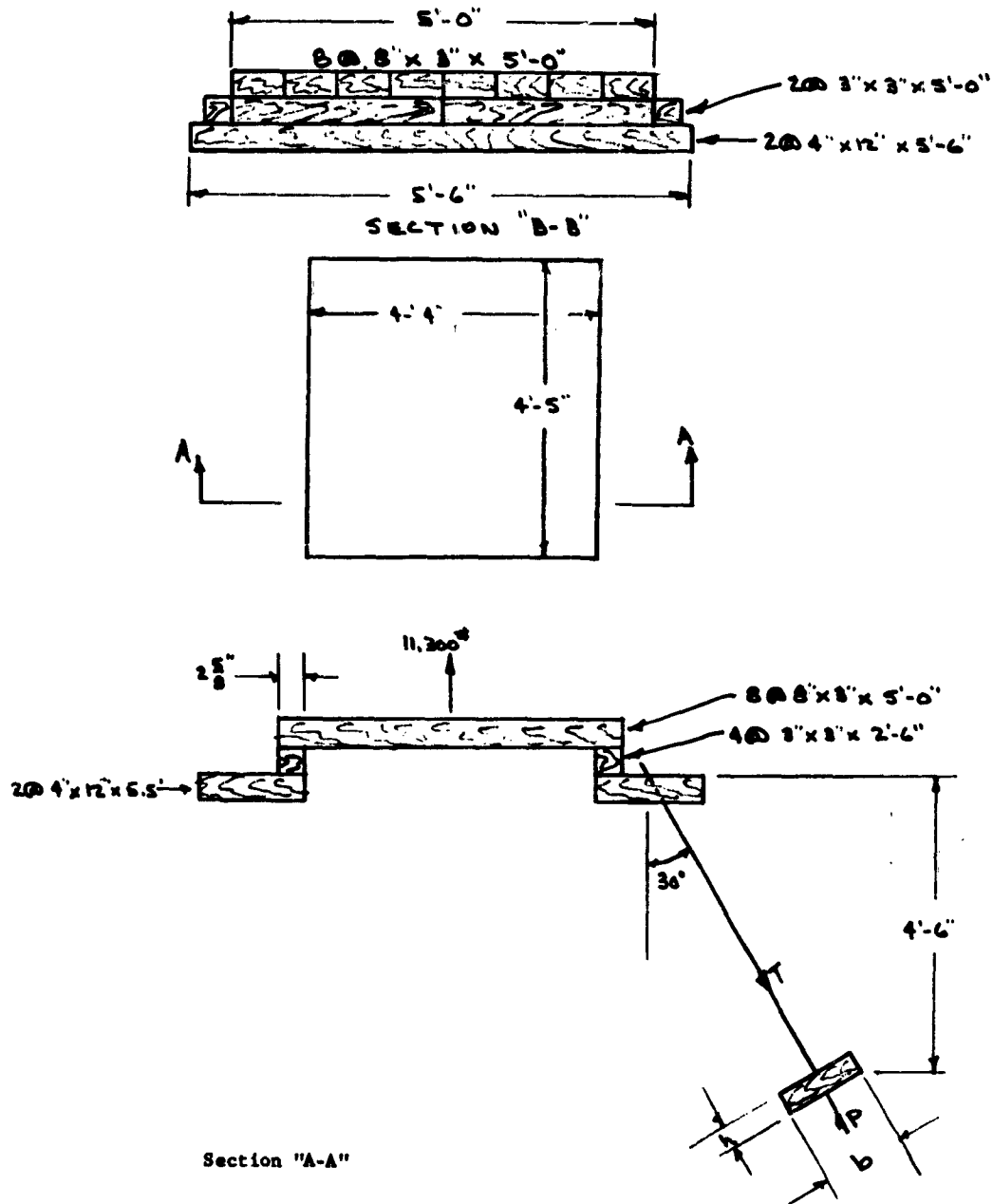
$P_g/P_{so} = -.13$

Max negative pressure $\quad P_g = -.13(20) = -2.6$ psi

Total uplift on cover $= P_g A = 2.6(5'-3'')(5'-2'')(144) \quad\quad 11,300$ lb

$\Sigma F_y = 0$

$2T \cos 30^\circ = 11,300$ lb

$$T = \frac{11,300}{2 \cos 30^\circ}$$

$$T = 6520 \text{ lb}$$

Soil $\quad \phi = 30^\circ \quad h' = 4.5' \quad L_s = 5.5' \quad P_{bear} = 3000$ psf

$$A_{min.} = \frac{6520}{3000} = 2.18 \text{ ft}^2$$

$$b_{min.} = \frac{2.18}{5.5} = .397 \text{ ft} = 4.76'' \quad\quad \text{Try } b = 5.5''$$
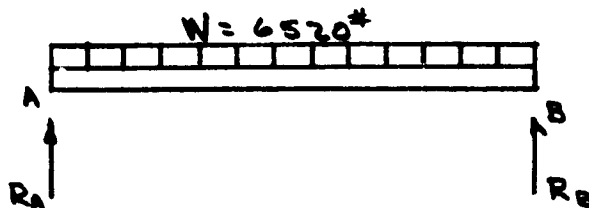


$$R_a = R_b = \frac{6520}{2} = 3260 \text{ lb}$$

$$M = \frac{WL}{8} = \frac{6520(5.5)(12)}{8} = 53,900 \text{ in. lb}$$

$$S = \frac{M}{f} = \frac{53,900}{6000} = 9 = \frac{bh^2}{6}$$

$$h^2 = \frac{9(6)}{5.5} = .982 \text{ in.}^2$$

$h = 3.22$ in. $\quad$ Try $h = 3.625$ in.

Shear

$$H = \frac{3V}{2bh} = \frac{3(3260)}{2(5.5)(3.625)} = 245 < 640$$

<u>Use 6" x 4" x 5'-6"</u>    wt = 27.5 lb     bm = 11     2 required

Tie Rods and Connections

$$R_A = 3265 \text{ lb} \qquad f = 50,000 \text{ psi (steel)}$$

$$A_S = \frac{R_A}{S} = \frac{3260}{50,000} = .0652 \text{ in.}^2$$

<u>Use Bar $\frac{5}{16}$ $\phi$ x 6' Lg</u>

$$C_\perp = 1100 \text{ psi}$$

$$A_W = \frac{3260}{1100} = 2.96 \text{ in.}^2 = \pi r^2$$

$$r = \frac{\sqrt{2.96}}{\pi}$$

$$r = .973 \text{ in.}$$

<u>Use 1" washer</u>
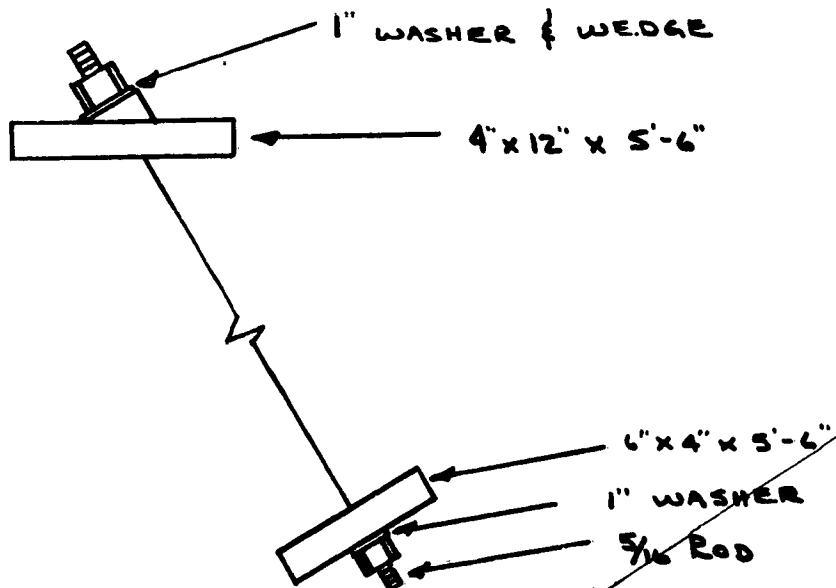
# APPENDIX H

## REINFORCED PLASTIC BOAT MANUFACTURERS IN THE UNITED STATES

| | |
|---|---|
| Aero Craft, Inc. | St. Charles, Mich. |
| Aeromarine Plastics Corp. | Sausalito, Calif. |
| Aero-Nautical Boat Shop | Copiague, L.I., N. Y. |
| Albright Boat & Marine Co. | Charlotte, N. C. |
| Alcort, Inc. | Waterbury, Conn. |
| Alexandria Boat Works, Inc. | Alexandria, Minn. |
| Allcock Mfg. Co. | Ossining, N. Y. |
| Allied Fibercraft | Brooklyn, N. Y. |
| All Star Boat Company | Lewisburg, Tenn. |
| Aluma Craft Boat Company | Minneapolis, Minn. |
| American Boat Building Corp. | East Greenwich, R. I. |
| American Molded Fiberglass Co. | Paterson 1, N. J. |
| The Anchorage, Inc. | Warren, R. I. |
| Anderson, H. H. Co. | Superior, Wisc. |
| Aqua Craft Ltd. | North Arlington, N. J. |
| Aquafleet | Allston, Mass. |
| Aqua Trailers, Inc. | Spokane, Wash. |
| Arena Boat Co. | Oakland 8, Calif. |
| Arrow Glass Boat & Mfg. Co. | Memphis, Tenn. |
| Arrowhead (Luck Boats) | Fort Worth, Tex. |
| Astra Industries, Inc. | Elgin, Ill. |
| Atlanta Boat Works | Atlanta, Ga. |
| Atlantic Marine Industries | Pemberton, N. J. |
| B & N Mfg. Co. | Kendallville, Ind. |
| Baker-Jewell | Orrville, Ala. |
| Balco Yacht Co. | Dundalk, Md. |
| Barracuda Marine Co. | Aurora, Ill. |
| Bauman-Harnish Rubber & Plastics | Garrett, Ind. |
| Baumann, G. H., Mfg. Co., Inc. | Amityville, N. Y. |
| Bee-Line Mfg. Co. | Grand Rapids, Mich. |
| Bellingham Shipyards Co. | Bellingham, Wash. |
| Bellboy Boats | Bellingham, Wash. |
| Bellboy Div. Lunn Laminates, Inc. | Huntington Sta., L.I., N. Y. |
| Bemidji Boat Co., Inc. | Bemidji, Minn. |
| Blue Mfg. Co., Inc. | Miami, Oklahoma |
| Boat Distribution, Inc. | Clinton, N. C. |
| Bock Boats, Inc. | Toledo, Ohio |
| Bo-Craft Plastics | Little Rock, Ark. |
| Bowman Mfgrs., Inc. | Lake Providence, La. |
| Bronson Boatbuilding Co. | Tacoma, Wash. |
| Brownie Boats | Avon Lake, Ohio |

| | |
|---|---|
| Bryant's Marina | Seattle, Wash. |
| Cape Cod Shipbuilding Co. | Wareham, Mass. |
| Carolina Fiberglass Products Co. | Wilson, N. C. |
| Central Boat Works | LaMarque, Tex. |
| Challenger Marine Corp. | North Miami, Fla. |
| Clipper Mfg. Co. | Fort Worth, Tex. |
| Comar Plastics Co., Inc. | Redwood City, Calif. |
| Coral Boat Co. | Nappanee, Ind. |
| Correct Craft, Inc. | Pinecastle, Fla. |
| Corsair Marine | Fort Worth, Tex. |
| Corson Boats | Madison, Me. |
| Creekmore Raymond | Miami 33, Fla. |
| Crestliner, Inc. | Strasburg, Va. |
| Cripe Equipment Co. | Wolcottville, Ind. |
| Crosby Aeromarine | Grabill, Ind. |
| Crown Line Mfg. Co. | Onarga, Ill. |
| Crystaliner Corp. | Newport Beach, Calif. |
| Customcraft, Inc. | Fort Wayne, Ind. |
| Custom Craft Marine Co. | Buffalo 7, N. Y. |
| Custom Fiberglass Products | Texarkana, Tex. |
| Customflex Industries | Toledo 14, Ohio |
| Cutter Boats, Inc. | Tell City, Ind. |
| Dallas Engineers, Inc. | Dallas, Pa. |
| Dawes Engineering | Sunnyvale, Calif. |
| Denison Plastics | Denison, Tex. |
|   (also: Yellow-Jacket Sales) | |
| Desert Marine, Inc. | Boise, Idaho |
| Dorsett Plastics Corp. | Santa Clara, Calif. - |
| | Bremen, Ind. |
| Dreamboat Mfg. Corp. | Guntersville, Ala. |
| Driscoll Brothers Boat Works | San Diego, Calif. |
| Duraglass Boat Co. | Monticello, Ark. |
| Duraplane Boats | Columbia, S. C. |
| Duratech Mfg. Corp. | Peekskill, N. Y. |
| Duo Cat | Fort Wayne, Ind. |
| Emigh Brothers Boat Works | Costa Mesa, Calif. |
| Enfab, Inc. | Santa Clara, Calif. |
| Fabuglas Co. | Nashville, Tenn. |
| Fabri-Glass | Moline, Ill. |
| Falls City Fiberglass | Louisville 15, Ky. |
| Feather Craft, Inc. | Atlanta, Ga. |
| Ferbend, S.U., Inc. | Morganfield, Ky. |
| Fiber Craft, Inc. | North Miami, Fla. |
| Fiber-Fab Co. | Dearborn 7, Mich. |
| Fiberglas Forms Industries | Twinsburg, Ohio |
| Fiber Glass Engineering, Inc. | Madison, Wisc. |
| Fiber-Resin Corp. | Burbank, Calif. |

| | |
|---|---|
| Fibra Glass Boat Co., Inc. | Waverly, Nebr. |
| Fisher Pierce Co. | Rickland, Mass. |
| Fleetcraft, Inc. | Woodbine, N. J. |
| Fleetcraft Marine Sales | Los Angeles, Calif. |
| Fleetform Corp. (Inboard Marine) | Fort Worth, Tex. |
| Florida Fiberglass Products, Inc. | Palatka, Fla. |
| Flyin' Flivver Co. | New Prague, Minn. |
| Fola Corp. | Lake Park, Minn. |
| Fort Dodge Boat Co., Inc. | Fort Dodge, Iowa |
| Frontier Fiberglas Industries | Cheyenne, Wyo. |
| Gad-A-Boat Mfg. Co. | Detroit 39, Mich. |
| Galbraith, C. C. & Son, Inc. | Keyport, N. J. |
| Gaycraft | Schofield, Wisc. |
| General Marine Co. | St. Joseph, Mo. |
| General Plastic Products Corp. | Anniston, Ala. |
| Geneva Boat Co. | Maitou Beach, Mich. |
| George D. O'Day Associates | Boston, Mass. |
| Gitt, E. & Sons | Springfield, Pa. |
| Glased Boat Co. | Fort Worth, Tex. |
| Glas-Bilt, Inc. | Fargo, N. D. |
| Glass Craft Boats, Inc. | Fort Dodge, Iowa |
| Glass Magic, Inc. | Fort Worth, Tex. |
| Glass Fiber Products, Inc. | Columbus, Ga. |
| Glassflite Co. | Houston, Tex. |
| Glass-Go Co. | Browns, Ala. |
| Glassmaster Plastics Co. | Columbia, S. C. |
| Glasspar Co. | Santa Ana, Calif. - Costa Mesa, Calif., and Petersburg, Va. - Nashville, Tenn. |
| | |
| Glastex Co. | Tinley Park, Ill. |
| Glastron Boat | Austin, Tex. - Madison, Ind. |
| Goodyear Aircraft Corp. | Akron, Ohio |
| Green, Ray, & Co. | Toledo, Ohio |
| Hands Shipbuilding Co. | Detroit 12, Mich. |
| Harvey Boat Works | Aloha, Ore. |
| Harwill, Inc. | St. Charles, Mich. |
| Henderson Plastic Engineer Corp. | Henderson, Ky. |
| Henry R. Hinckley & Sons | Southwest Harbor, Me. |
| Herters | Waseca, Minn. |
| Hickel Plastic Products | Indianapolis, Ind. |
| Holiday Plastics, Inc. | Kansas City, Kans. |
| Howard Boat Mfgrs. | Wrentham, Mass. |
| Hupp Engr. Association | Bloomington, Ill. |
| Hurricaw & Boat Co. | Houston, Tex. |
| Ideal Aerosmith, Inc. | Hawthorne, Calif. |
| Imperial Boat Co. | Auburn, Me. |

| | |
|---|---|
| Indiana Gear | Indianapolis, Ind. |
| Indianapolis Wire Bound Box | Indianapolis, Ind. |
| International Yacht Sales | Detroit, Mich. |
| Invader Mfg. Corp. | Fort Worth, Tex. |
| Jayhawk Marine, Inc. | Parsons, Kans. |
| Kenner Boat Co., Inc. | Knoxville, Ark. |
| Kerrco Products | Lincoln, Nebr. |
| Kettenburg Boat Co. | San Diego, Calif. |
| Laby Engineering Corp. | Van Nuys, Calif. |
| Lake Aire Marine, Inc. | Clear Lake, Iowa |
| Lane Lifeboat & Davis Corp. | Brooklyn, N. Y. |
| Larsen Marina | Burton, Wash. |
| Larson Boat Works, Inc. | Nashville, Ga. |
| Lee Craft Marine | Somers, Mont. |
| Lofland Co. | Wichita, Kans. |
| Lone Star Boat Co. | Grand Prairie, Tex. - |
| | McAddo, Pa., and |
| | Bremen, Ind. |
| | |
| Luders Marine Construction Co. | Stamford, Conn. |
| Lunn Laminates, Inc. | Huntington Sta., L.I., N. Y. |
| LuTo Co., Inc. | Salt Lake City, Utah |
| Lynx Fiberglas Boats | Los Gatos, Calif. |
| Magnolia Boat Mfg. Co. | Vicksburg, Miss. |
| Mair's Marine Mart, Inc. | Minocqua, Wisc. |
| Marine Fiber-Glass & Plastics, Inc. | Seattle 5, Wash. |
| Marine Plastics, Inc. | Fort Worth, Tex. |
| Marlin Boat Co. | Fort Myers, Fla. |
| Marlin Fiberglass Boat Co. | Boca Raton, Fla. |
| Marscot Boats, Inc. | Jacksonville 5, Fla. |
| Marscot Plastics, Inc. | New Bedford, Mass. |
| Merline Boats | Downey, Calif. |
| Metcalf | Costa Mesa, Calif. |
| Meyers Marine, Inc. | Columbia City, Ind. |
| Miami Marine Industries | Little Rock, Ark. |
| Modular Molding Corp. | Burlington & Trenton, N. J. |
| Molded Fiber Glass Boat Co. | Union City, Pa. |
| Multi-Plastics Co. | Fort Worth, Tex. |
| Myco Marine | Belvedere, Ill. |
| National Marine Plastics | Tulsa, Okla. |
| Newsome, Jake, Boats | Bradenton, Fla. |
| North American Marine, Inc. | Warsaw, Ind. |
| Northwest Mfg. Corp. | Iron River, Wisc. |
| Nylox Corp. | Arcadia, Calif. |
| Octopus Boat & Plastic Co. | Miami 33, Fla. |
| Orlando Boat Co. | Orlando, Fla. |
| Orrcraft Mfg. Co. | Orville, Ohio |
| Owens Yacht Co., Inc. | Baltimore 22, Md. |

| | |
|---|---|
| P-14 Fiberglass Boat | Pilot Grove, Mo. |
| Pabst Boats, Inc. | Tomahawk, Wisc. |
| Pacific Fiber Glass & Boat Co., Inc. | San Pedro, Calif. |
| Pacific Plastic Co., Inc. | Seattle, Wash. |
| Parsons Corp. | Traverse City, Mich. |
| Patterson Boats | North Carolina |
| Pearson Corp. | Bristol, R. I. |
| Pere Marquette Fiberglass Boat Co. | Scottville, Mich. |
| Perma Craft Corp. | Hollywood, Fla. |
| Perma Craft Products Co. | Fullerton, Calif. |
| Pipestone Sales, Co. | Pipestone, Minn. |
| Plas-Steel Products, Inc. | Walkerton, Ind. |
| Plastic Fabrications, Inc. | Hialeah, Fla. |
| Plastic Industries, Inc. | Independence, Mo. |
| Plastaglass Co. | Newport Beach, Calif. |
| Plastikaire Products, Inc. | Westbury, L.I., N. Y. |
| Plastyle Co., Inc. | Niles, Mich. |
| Polymer Engineering Corp. | Houston, Tex. |
| Power Cat Boat Corp. | Paramount, Calif. |
| Progressive Plastic Prods., Inc. | San Antonio, Tex. |
| Quality Plastic, Inc. | Reeseville, Wisc. |
| Ratio Mfg. Co. | Navasota, Tex. |
| Red Fish Boat Co., Inc. | Clarksville, Tex. |
| Red Wing Fiberglass Industries | Red Wing, Minn. |
| Reinell Boat Works | Marysville, Wash. |
| Repco, Inc. | Jefferson, Mass. |
| Rocket Marine, Inc. | El Monte, Calif. |
| Rose Fiberglass Boat Co. | Knoxville, Tenn. |
| Royalcraft Boat Co. | Hanover, N. J. |
| Sabre Craft Boat Co., Inc. | Seattle, Wash. |
| Salerno Shipyard, Inc. | Salerno, Fla. |
| Schenkel Brothers Mfg. Co. | Brookville, Ind. |
| Scottie Craft Boat Mfg., Inc. | Hialeah, Fla. |
| Seamaid Mfg. Co., Inc. | Kendallville, Ind. |
| Sea Otter Boat Co., Inc. | Chicago, Ill. |
| Sears, Roebuck & Co. | Chicago, Ill. |
| Sea Sled Industries, Inc. | Skokie, Ill. |
| Sea Fury, Inc. | Fort Lauderdale, Fla. |
| Shawnee Plastic Co., Inc. | Chester, Ill. |
| Shell Lake Boat Co. | Shell Lake, Wisc. |
| Silver Star Fiberglass Prods., Inc. | Wolcottville, Ind. |
| Skagit Plastics | LaConner, Wash. |
| Sooner Boat Co. | El Dorado, Okla. |
| S. Bend Div. of S. Bend Laminated Prod., Inc. | South Bend, Ind. |
| South Coast Co. | Newport Beach, Calif. |
| Southern Mfg. Co. | Daytona Beach, Fla. |

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| Southwest Mfg. Co. | Little Rock, Ark. - Amsterdam, N. Y., and Adams, Wisc. |
| Span American Boat Co., Inc. | Fort Dodge, Ia. |
| Specht Plastic Co. | Somerset, Pa. |
| Spore Boat Co. | Princeton, Ind. |
| Stamm Boat Co. | Delafield, Wisc. |
| Starcraft Boat Co. | Goshen, Ind. |
| Steury Boat Co. | Goshen, Ind. |
| Storecrafters, Inc. | Houston, Tex. |
| Su-Mark, Inc. | Walpole, Mass. |
| Sumner Boat Co., Inc. | Amityville, L.I., N. Y. |
| Superglas Corp. | Nashville, Tenn. |
| Superior Plastics Co. | Detroit, Mich. |
| Texas Boat Mfg. Co., Inc. | Lewisville, Tex. |
| The Laurel Corp. | Shippenville, Pa. |
| Tomahawk Boat Mfg. Corp. | Tomahawk, Wisc. |
| Trail-It Coach Mfg. Co., Inc. | Des Moines, Ia. |
| Trailorboat Engineering Co. | San Rafael, Calif. |
| United Boatbuilders, Inc. | Bellingham, Wash. |
| U. S. Fiber Glass Products, Inc. | Paramount, Calif. |
| U. S. Plastics of Florida, Inc. | Fort Lauderdale, Fla. |
| Universal Moulded Products Corp. | Bristol, Va. |
| Utility Plastics Co. | Tulsa, Okla. |
| Viking Boat Co. | Middlebury, Ind. |
| Viking Reinforced Plastics | Elbow Lake, Minn. |
| Vitale Plastics | Long Island City, N. Y. |
| Venentian Marina | Miami Beach, Fla. |
| Vio Holda Mfg. Co. | Topeka, Kansas |
| W. D. Schock | Newport Beach, Calif. |
| W. R. Chance & Associates | Waldorf, Md. |
| Wacanda Marine | Colville, Wash. |
| Wagemaker Co. | Grand Rapids, Mich. |
| Wells Fiberglass Sailboats | Houston, Tex. |
| Westerner Boat | South Gate, Calif. |
| Western Textile Co. | St. Louis, Mo. |
| Whitehouse Reinforced Plastic Co. | Fort Worth, Tex. |
| Wilco Products | Mishawak, Ind. |
| Wilmar Boat Co., Inc. | Richmond, Ind. |
| Wilmar Distributing Co. | Pasadena 8, Calif. |
| Wizard - Winner Boats of Tenn. | Dickson, Tenn. |
| Wizard Boats, Inc. | Costa Mesa, Calif. |
| Wright, John Jr. | Philadelphia, Pa. |
| Zenith Plastics Co. | Gardena, Calif. |

AD

Accession No.

U. S. Army Engineer Research and Development Laboratories, Fort
Belvoir, Virginia - GROUP SHELTER INVESTIGATION - Richard M. Flynn

Unnumbered Report, 29 Oct 62, 316 pp, 35 illus, 16 tables
DA Task 8A72-04-001-31, Office of Civil Defense Project 1702

Unclassified Report

Office of Civil Defense Project 1702 requires that investigation
be made into the possibilities of designing group shelter elements
to enable unskilled people with only light equipment to erect low-
cost group shelters. Work on project requirements was divided into
four parts concomant with materials: timber, metal, concrete, and
plastics. Work on timber, metal, and concrete was performed at
USAERDL. Work on plastic was performed under contract with DeBell
and Richardson, Inc., Enfardville, Connecticut. The report con-
cludes: (a) It is possible to design and successfully develop group
shelters that can be erected by unskilled groups of people;
(b) such shelters can be erected with only hand tools and easily
constructed expedient equipment such as "A" frames; (c) ordinary
engineering materials can be used in the shelters' construction;
(d) material cost based on a 60-man shelter will not exceed about
$80 per man; (e) further investigation and field testing will be
necessary to fully and accurately evaluate construction techniques
and costs relative to a specific design; (f) development of plastic
design 1 is desirable since it offers the potentiality of the most
simple construction techniques together with costs comparable to
the other engineering materials.

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